WEEK2

TRAFFIC FLOW: THEORY AND ANALYSIS

2016年9月12日

§2. Traffic flow

interact → Drivers
→ Vehicles
→ Infra

types { uninterrupted flow
interrupted flow

Microscopic vs. Macroscopic
Average speed
Density

---

1. Macroscopic:

Water flow vs. Traffic flow

Road types { highway
Arterial
Residential.

Traffic peak volume

Vehicle speed

Key parameters

① Volume / Flow rate { Volume. (observed)
Rate of flow: veh/hr (projected) ⇒ how many

② Speed.

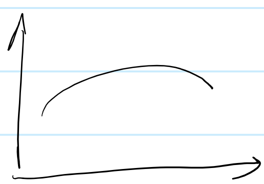
③ Density

Daily volumes (veh/day)

- AADT $\dfrac{\text{total 24h' volume within one year}}{\text{365 days}}$
- AAWT $\dfrac{\text{24h' volume within one year}}{\text{260 days}}$
- ADT $\dfrac{\text{24h' volume a period (2–365) days}}{\text{a period}}$
- AWT $\dfrac{\text{24h' volumes a period}}{\text{a period}}$.

⟹ $\boxed{\text{for planning purpose.}}$

Demand trend.



Accidents

+

2

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– AADT:	average annual daily volume
– AAWT:	average annual weekday volume
– ADT:	average daily volume
– AWT:	average weekday volume

Hourly volume: $\Rightarrow$ $\boxed{\text{Design}}$

– DDHV: directional design hourly volume.

$$DDHV = AADT \times K \times D$$

K – propotion of peak hour volume within the daily volume

D – directional factor

ADT: 9100 v/D

Directional split: 60/40

Ph Volume: 1138 vph

$$\underbrace{9100}_{ADDT} \times \underbrace{\frac{1138}{9100}}_{K} \times \underbrace{\frac{60}{60+40}}_{D}$$

Sub-hourly volume: PHF – Peak hour factor

$\Rightarrow$ opreation ex. signal timing

– PHF

$$PHF = \frac{V}{(60/15) \times V_{15}} = \frac{V}{4V_{15}}$$

$4V_{15} \rightarrow$ rate of flow

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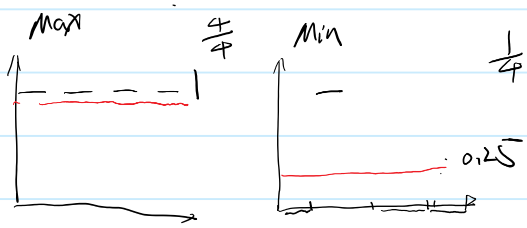
Implication

≈ 1

maximum & Minimum Value



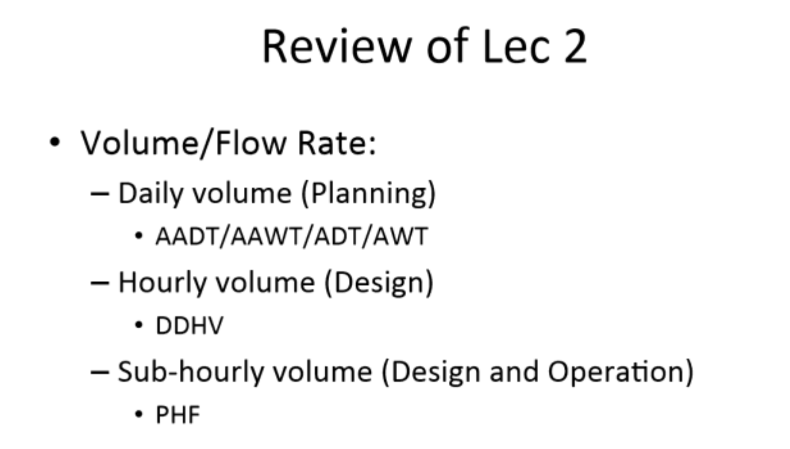
Volume data collection

loop detector
Microwave detector
counter

Wis Transportal.

Review of Lec2

- Volume/Flow Rate:
` – Daily volume (Planning)
- AADT/AAWT/ADT/AWT
– Hourly volume (Design)
- DDHV
– Sub--hourly volume
(Design and Operalon)
- PHF



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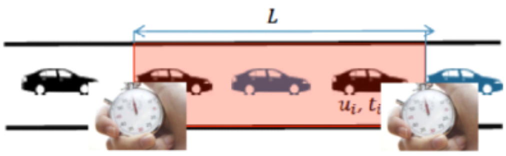
Speed

- To reflect the traffic conditions
- Speed limit enforcement (safety)
- To establish the basis for design of speed limit

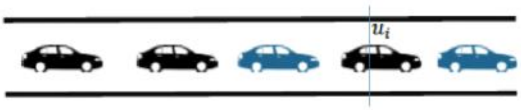
Time mean speed (TMS)	Average speed of all vehicles passing a point over some specified time period
Space mean speed (SMS)	Average speed of all vehicles occupying a given section over some specified time period

Traffic counter



TMS: $\overline{u_t} = \frac{\sum_{i=1}^{n} u_i}{n}$ Arithmetic Mean $u = \frac{L}{t}$

⇒ individual vehical

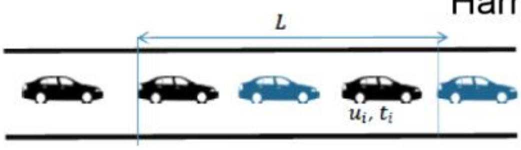


Traffic counter

(MPH)☆

SMS: $\overline{u_s} = \frac{L}{t_{avg}} = \frac{L}{\frac{\sum_{i=1}^{n} t_i}{n}} = \frac{nL}{\sum_{i=1}^{n} t_i} = \frac{n}{\sum_{i=1}^{n}(1/u_i)}$

Harmonic Mean



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usually, TMS ≥ SMS

Approximate Relationship Between SMS & TMS (Wardrop, 1952)

$$v_t = v_s + \frac{\sigma^2_s}{v_s}$$ $$\sigma^2_s = [\sum (v_i - v_s)^2]/n$$

Approximate Relationship Between SMS & TMS (Wardrop, 1952)

$$v_t = v_s + \frac{\sigma^2{}_s}{v_s}$$

$$\sigma^2{}_s = [\sum (v_i - v_s)^2]/n$$

Percentile Speed ⇒ how fast

Speed Limit

- The design of speed limit in the Manual on Uniform Traffic Control Devices (MUTCD) use 85 percentile speeds
- 20 speeds are taken:

  50, 46, 44, 42, 42, 41, 41, 41, 40, 40, 40, 40, 39, 39, 37, 37, 36, 36, 36, 35
- 85th Percentile Speed from bottom is 42.3 mph (go to D2L and download the excel file)
- What speed limit should be set? ⇒ 40 MPH

Percentile (A1:A20, 0.85)

Density ⇒ how dense

- Density
  - Veh/mile/lane
  - How to calculate?

*<u>**Density**</u>: Number of vehicles occupying a given length of lane or roadway, averaged over time, usually expressed as vehicles per mile per lane*

veh

$$D = \frac{42}{\frac{576'}{5280} \times 4}$$

mile

lanes

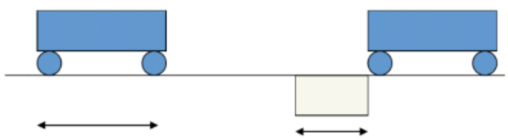
- Occupancy
- Density is difficult to measure. So, we use "occupancy" as a surrogate measure for density. This can be obtained by traffic detectors of any kind.
- Occupancy: the percent of the roadway (in terms of time) that is covered (occupied) by vehicles. (UNIT: %)

Example: A loop detector was occupied for 45 sec within 1-min time interval, what is the O?

- Occupancy (O) and Density (D)

$$D = \frac{5280 * O}{L_v + L_d}$$

Hint: considering the situation when O=100%



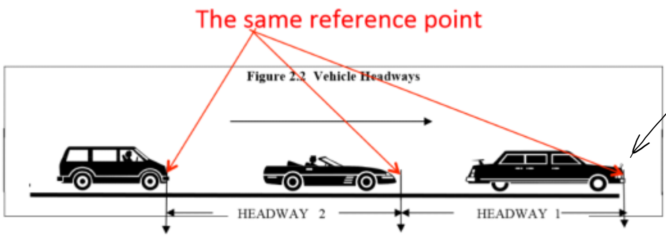
Lv: the length of vehicle (unit: ft)

Ld: the length of detector (unit: ft)

Microscopic

Speed
headway
Spacing

Headway & Spacing



front bumper

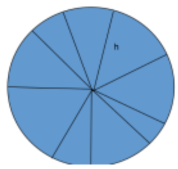
- Time Headway, "Headway"
- Distance Headway, "Spacing"

headway:

inter-arrival time between two consecutive vehicals.

Microscopic 	Macroscopic $q = D \times S$ 
Headway (or time headway), h	*Volume or flow rate q,* $q = 3600/\bar{h}$;
Speeds of individual vehicles, u_i	Speed $S = (\bar{d}/\bar{h})/1.47$ $TMS = \sum_{i=1}^{N} u_i / N$ $SMS = N / \sum_{i=1}^{N} 1/u_i$
Spacing (or distance headway), d	Density, D = 5280/$\bar{d}$ *Occupancy,* $D = \frac{5280 * O}{L_v + L_d}$

- Headway (in secs) → Flow rate



\# of vehicles in 1 hour

$q = 3600 / \bar{h}$ (1)

$\Rightarrow D = \frac{1\,mile}{\bar{d}} = \frac{5280\,ft}{\bar{d}}$ (2)

Fundamental Equation

- Flow=Density*Space Mean Speed
- q=D*S

$\frac{q}{D} = \frac{veh/hour}{veh/mile} = \frac{mile}{hour} = S$ (3) m/h

put (1) (2) into (3)

$\Rightarrow S = \frac{(\bar{d}/\bar{h})}{1.47}$ ft/sec

- The leftmost lane on a 6-lane freeway has a density of 45 vehicles per lane-mile and a speed of 50 mph. What is the flow rate ⇒ q? per lane, the average headway and the average spacing? $\bar{h}$ = ? $\bar{d}$ = ?

6-lane.

$D = 45$ v/l/mile

$S = 50$ mph

$q = ?$

$q = D \cdot S = 45 \times 50 = 2250$

$\bar{h} = \frac{3600\,hrs}{q} = \frac{3600}{2250} = 1.6$ sec

$\bar{d} = \frac{5280\,ft}{D} = \frac{5280}{45} = 117.3$ ft

- The leftmost lane on a 6-lane freeway has a density of 45 vehicles per lane-mile and a speed of 50 mph. What is the flow rate ⇒ q per lane, the average headway and the average spacing? $\bar{d}=?$ $\bar{h}=?$

6-lane

$D = 45$ v/l/mile

$S = 50$ mph

$q = ?$

$q = D \cdot S = 45 \times 50 = 2250$

$\bar{h} = \frac{3600 \text{ hrs}}{q} = \frac{3600}{2250} = 1.6$ sec

$\bar{d} = \frac{5280 \text{ ft}}{D} = \frac{5280}{45} = 117.3$ ft

$$\Rightarrow S = \frac{\left(\bar{d}/\bar{h}\right)}{1.47} \quad \text{ft/sec}$$

Traffic Flow Theory II:
• Greenshield's Model
• Stochastic Models

3-WEEK

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Greenshield's Model

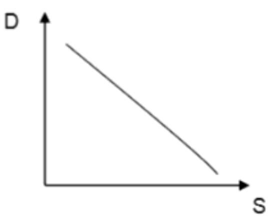
- I. Fundamental Equation

$$q = D \times S$$

- II. Speed density relation

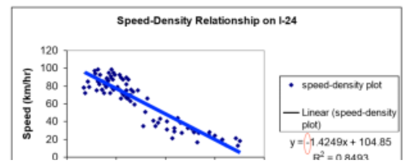
hypothesis: Linear

1) S—D,

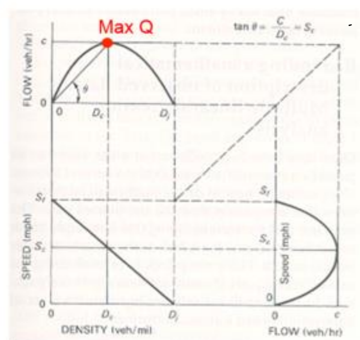


inverse proportional relationship

Speed-Density



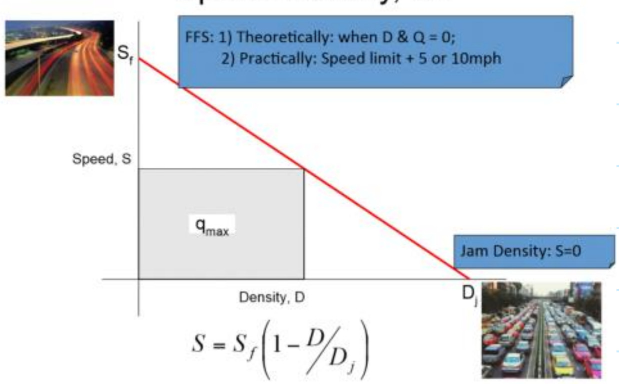
Greenshield's Model



2) Q—D: second Degree parabola

3) S—Q:

Greenshield's Model, Notation, Speed- Density, SD



$$S = S_f\left(1 - D/D_j\right)$$

S_f: max speed. FFS: Free-flow speed. D & $Q = 0$

D_j: max density $S = 0$

$$q = S \times D$$
$$= S_f\left(1 - D/D_j\right) \times D$$

$\Rightarrow q = S_f\left(D - \frac{D^2}{D_j}\right)$ ② constant variable

$\Rightarrow D_{q_{max}} = \frac{D_j}{2}$ ④

$\Rightarrow q = D_j\left(S - \frac{S^2}{S_f}\right)$ ③

$\Rightarrow S_{q_{max}} = \frac{S_f}{2}$ ⑤

④ → ②
⑤ → ③ $\Rightarrow q_{max} = \frac{D_j \cdot S_f}{4}$

02 II - Stochastic Traffic

2016年9月19日 8:43

- Brief Review of Probability Theory
- Poisson/Exponential Distributions
- Poisson Applications

Definition of Probability

- Probability is the measure of the likeliness that an event *will* occur
- Call this P[x], x stands for an event.

$$P[x] = \frac{\text{The number of favorable outcomes}}{\text{The total number of possible outcomes}}$$

Poisson

- Discrete distribution
- Commonly referred to as '*counting distribution*'
- Represents the count distribution of random events

Example: the number of cars passing a specific point in one hour

Poisson Mass Function (Eq 2.24)

$$P[n] = \frac{(\lambda t)^n e^{-\lambda t}}{n!}$$

Time Units!

P[n] = probability that exactly n vehicles arrive in time interval t

t = duration of time interval (sec, min, hr, etc)

λ = average flow rate or arrival rate (vehicles/ time)

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Determine headway

Exponential Distribution, Time between Random Events (Headway!)

$$P(h > t) = e^{-\lambda t}$$

$$P(h<t) = 1 - P(h>t) = 1 - e^{-\lambda t}$$

h = headway between two vehicles

t = time duration in which 0 vehicles will arrive (sec, min, hr, etc)

λ = average flow rate or arrival rate (vehicles/ time)

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Left-turn storage length

- Want to design left-turn bay length to accommodate all of the LT vehicles a certain % of the time
- Based on ? volumes

Hourly

AADT Daily

DDHV Hourly

PHF Sub hourly

headway > t

WEEK 4

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Highway Capacity - Freeways

Outline
1. Definitions
2. Level of Service (LOS)
3. Freeway Segment LOS

a. Free-flow speed	b. Flow Rate c. LOS

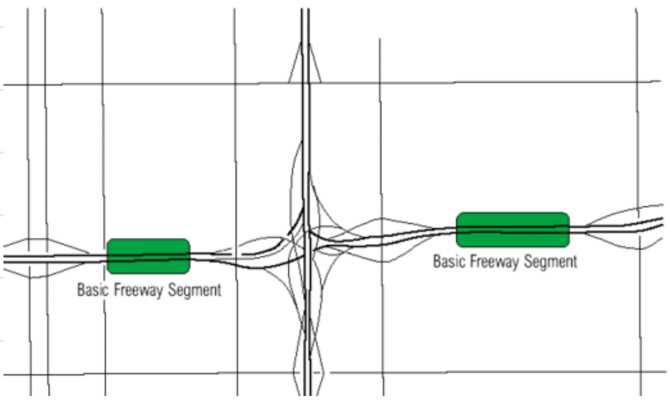
1、

Definitions – Freeway

- A “controlled-access” highway/express highway

2、

Freeway Elements



Definitions – freeway capacity

Freeway capacity (unit: pc/ln/hr or pcplp)

- the maximum sustained 15-min flow rate, expressed in passenger cars per hour per lane, that can be accommodated by a uniform freeway segment under prevailing traffic and roadway conditions in one direction of flow.

not include heavy vehichles

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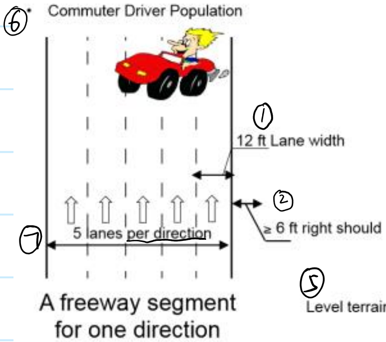
Ideal Freeway (7 criteria)

③ All Passenger Car Composition

⑥ Commuter Driver Population



⑤ Level terrain, grades < 2%

Ideal Freeway

① • 12-foot lanes
② • Paved right shoulder ≥ 6 feet
③ • Passenger cars only
④ • Interchanges ≥ 2 miles
⑤ • Level terrain, grades < 2%
⑥ • Driver population dominated by regular/familiar users
⑦ • 10-lane freeway or greater (in urban areas)

4、

Ideal Capacity

- Freeways: Capacity

  2,400 pcphpl (70 mph)
  2,350 pcphpl (65 mph)
  2,300 pcphpl (60 mph)
  2,250 pcphpl (55 mph)

5、

Definitions – Free-Flow Speed

- Free-Flow Speed (FFS)
  - The mean speed of passenger cars that can be accommodated under low to moderate flow rates on a uniform freeway segment under prevailing roadway and traffic conditions.
- Factors affecting free-flow speed
  - Lane width
  - Lateral clearance
  - Number of lanes
  - Interchange density
  - Geometric design

Definitions– Free-Flow Speed (Cont'd)

- Passenger car equivalents
  - Trucks and RVs behave differently
  - Baseline is a freeway with all passenger cars
  - Traffic is expressed in passenger cars per lane per hour (pc/ln/hr or pcplph)
- Driver population
  - Non-commuters suck more at driving
  - They may affect capacity
- Capacity
  - Corresponds to LOS E and v/c = 1.0

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Definitions – Level of Service (LOS)

- Chief measure of "quality of service"
  - Describes operational conditions within a traffic stream.
- Six measures (A through F)
  - See LOS in action (Video)
- Freeway LOS
  - Based on traffic density

Measurement:

Density

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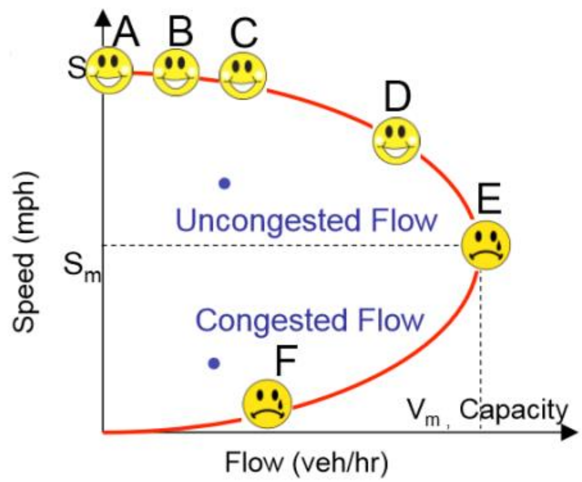
Definitions – Level of Service (LOS)

Level of Service	Density (passenger car/mi/ln)
A	≤ 11
B	> 11 - ≤ 18
C	> 18 - ≤ 26
D	> 26 - ≤ 35
E	> 35 - ≤ 45
F	> 45

Traffic Flow Characteristics

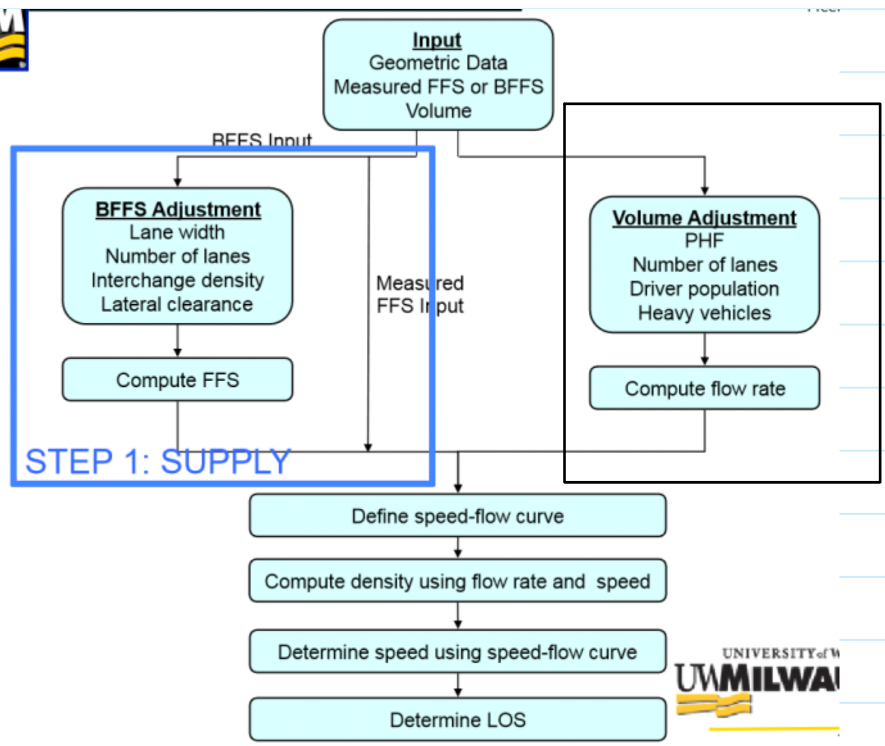


Source: Highway Capacity Manual (HCM), 2000

LOS-E: V/C=1 ⇒ maximum Capacity

LOS Determination.

BFFS — Ideal freeway



Demand.

Determining FFS

- Measure FFS in the field
  - Low to moderate traffic conditions
- Use a baseline and adjust it (BFFS)

$$FFS = BFFS - f_{LW} - f_{LC} - f_N - f_{ID}$$

FFS = free-flow speed (mph)
BFFS = base free-flow speed, 70 mph (urban), 75 mph (rural)
f_{LW} = adjustment for lane width (mph)
f_{LC} = adjustment for right-shoulder lateral clearance (mph)
f_N = adjustment for number of lanes (mph) → 0.0 for rural free way segment
f_{ID} = adjustment for interchange density (mph)

Lane Width Adjustments

- Base Conditions: 12 foot lanes

EXHIBIT 23-4. ADJUSTMENTS FOR LANE WIDTH

Lane Width (ft)	Reduction in Free-Flow Speed, f_{LW} (mi/h)
12	0.0
11	1.9
10	$\boxed{6.6}$

Lateral Clearance Adjustments

- Base Conditions: 6 feet right-shoulder lateral clearance

EXHIBIT 23-5. ADJUSTMENTS FOR RIGHT-SHOULDER LATERAL CLEARANCE

Right-Shoulder Lateral Clearance (ft)	Reduction in Free-Flow Speed, f_{LC} (mi/h) — Lanes in One Direction: 2	3	4	≥ 5
≥ 6	0.0	0.0	0.0	0.0
5	$\boxed{0.6}$	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

Number of Lanes Adjustments

- Base condition
  - 5 or more lanes in one direction
  - Do not include HOV lanes
  - $f_N = 0$ for all rural freeway segments

EXHIBIT 23-6. ADJUSTMENTS FOR NUMBER OF LANES

Number of Lanes (One Direction)	Reduction in Free-Flow Speed, f_N (mi/h)
≥ 5	0.0
4	$\boxed{1.5}$
3	3.0
2	4.5

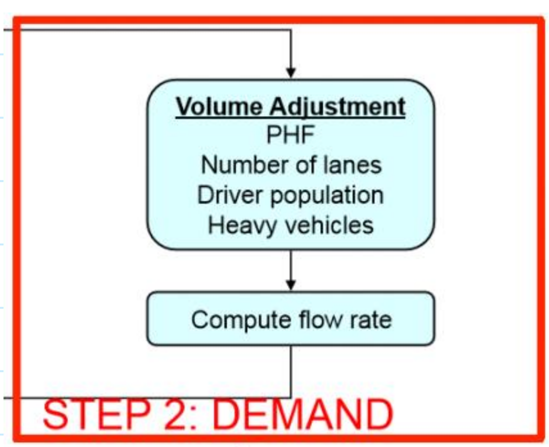
Note: For all rural freeway segments, f_N is 0.0.

Interchange Density Adjustments

- Base Conditions: interchanges spacing: 2 miles

EXHIBIT 23-7. ADJUSTMENTS FOR INTERCHANGE DENSITY

Interchanges per Mile	Reduction in Free-Flow Speed, f_{ID} (mi/h)
0.50	0.0
0.75	1.3
1.00	$\boxed{2.5}$
1.25	3.7
1.50	5.0
1.75	6.3
2.00	7.5



Determining Flow Rate

- Adjust hourly volumes to get pc/ln/hr

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

v_p = 15-minute passenger-car equivalent flow rate (pcphpl)
V = hourly volume (veh/hr)
PHF = peak hour factor
N = number of lanes in one direction
f_{HV} = heavy-vehicle adjustment factor
f_p = driver population adjustment factor

Peak Hour Factor (PHF)

- Typical values
  - 0.80 to 0.95
  - Lower PHF characteristic or rural or off-peak
  - Higher PHF typical of urban peak-hour

$$PHF = \frac{V}{V_{15} \times 4}$$

V = hourly volume (veh/hr) for hour of analysis
V_{15} = maxiumum 15-min. volume within hour of analysis
4 = Number of 15-min. periods per hour

Heavy Vehicle Adjustment (f_{HV})

- Base condition (f_{HV} = 1.0)
  - No heavy vehicles
  - Heavy vehicle = trucks, buses, RVs
- Two-step process
  - Determine passenger-car equivalents (E_T, E_R)
  - Determine proportion of heavy vehicles (P_T, P_R)

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \quad (23\text{-}3)$$

where
E_T, E_R = passenger-car equivalents for trucks/buses and recreational vehicles (RVs) in the traffic stream, respectively;
P_T, P_R = proportion of trucks/buses and RVs in the traffic stream, respectively; and
f_{HV} = heavy-vehicle adjustment factor.

Driver Population Adjustment (f_P)

- Base condition ($f_P = 1.0$)
  - Most drivers are familiar with the route
    - Commuter drivers: 1.0
  - Typical values between 0.85 and 1.00

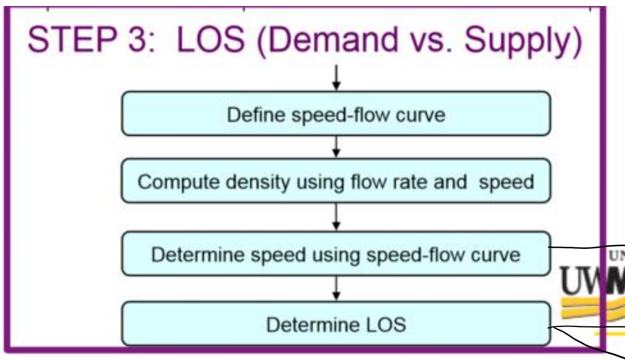
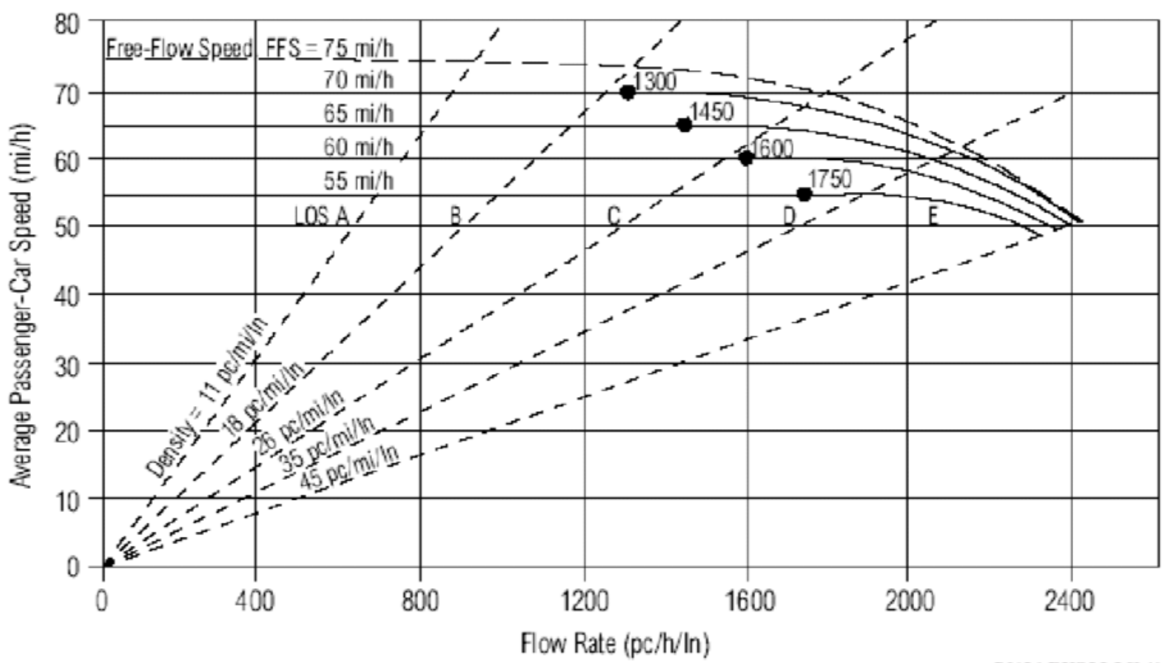


chart
Equation.
chart
Density

Define Speed-Flow Curve

Select a Speed-Flow curve based on FFS

EXHIBIT 23-3. SPEED-FLOW CURVES AND LOS FOR BASIC FREEWAY SEGMENTS



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Determine Density

- Calculate density using:

$$D = \frac{v_p}{S}$$

LOS	Density Range (pc/mi/ln)
A	0-11
B	>11-18
C	>18-26
D	>26-35
E	>35-45
F	>45

D = density (pc/mi/ln)
v_p = flow rate (pc/hr/ln)
S = average passenger-car speed (mph)

Determine Average PC Speed (S)

- For 70 < FFS ≤ 75 mph AND (3400 – 30FFS) < v_p ≤ 2400

$$S = FFS - \left[\left(FFS - \frac{160}{3}\right)\left(\frac{v_p + 30FFS - 3400}{30FFS - 1000}\right)^{2.6}\right]$$

- For 55 < FFS ≤ 70 mph AND (3400 – 30FFS) < v_p ≤ (1700 + 10FFS)

$$S = FFS - \left[\frac{1}{9}(7FFS - 340)\left(\frac{v_p + 30FFS - 3400}{40FFS - 1700}\right)^{2.6}\right]$$

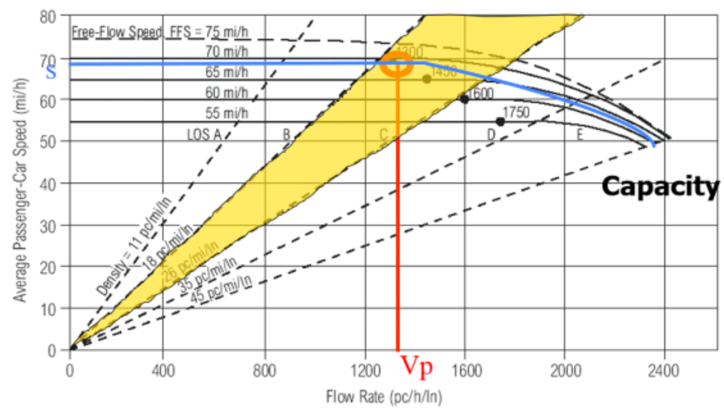
- For 55 < FFS ≤ 75 mph AND v_p < (3400 – 30FFS)

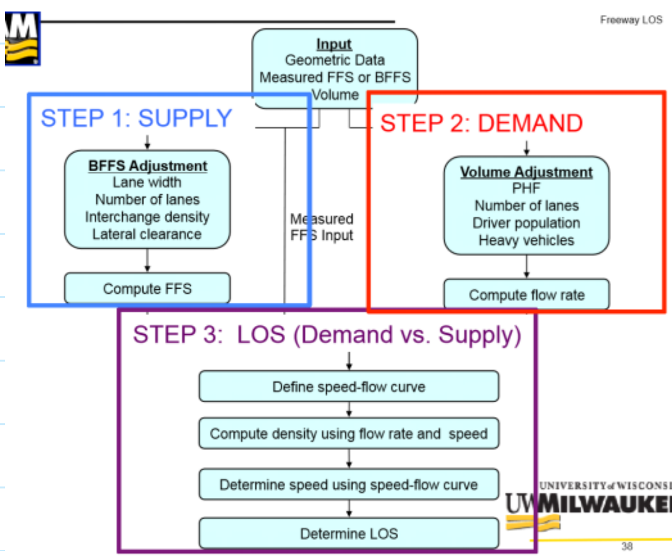
$$S = FFS$$

An easy way: Determine LOS using the chart in Exhibit 23-3

EX: Urban freeway segment. Vp = 1300 pc/h/ln and FFS = 68 mi/h
LOS=?

EXHIBIT 23-3. SPEED-FLOW CURVES AND LOS FOR BASIC FREEWAY SEGMENTS





STEP 1: SUPPLY

$$FFS = BFFS - f_{LW} - f_{LC} - f_N - f_{ID}$$

FFS = free-flow speed (mph)
BFFS = base free-flow speed, 70 mph (urban), 75 mph (rural)
f_{LW} = adjustment for lane width (mph)
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f_N = adjustment for number of lanes (mph)
f_{ID} = adjustment for interchange density (mph)

0.0; for rural freeway segment

STEP 2: DEMAND

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

v_p = 15-minute passenger-car equivalent flow rate per lane (pcphpl)
V = hourly volume (veh/hr)
PHF = peak hour factor
N = number of lanes in one direction
f_{HV} = heavy-vehicle adjustment factor
f_p = driver population adjustment factor

1.0; for commuter traffic

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

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Operation

Example I – Evaluation Problem

The Freeway Existing four-lane freeway, rural area, very restricted geometry, rolling terrain, 75 mi/h speed limit.

The Question What is the LOS during the peak hour?

The Facts

- N √ Two lanes in each direction,
- L_w √ 11-ft lane width,
- L_c √ 2-ft lateral clearance,
- $f_p=1$ √ Commuter traffic,
- V √ 2,000-veh/h peak-hour volume (one direction),
- √ 5 percent trucks,
- PHF √ 0.92 PHF,
- ID = 1 √ 1 interchange per mile, and
- $P_{hv}=5\%$ √ Rolling terrain.

Step 1: SUPPLY – Calculating FFS

Step 2: DEMAND – Calculating Vp

Step 3: EVALUATION – Determining LOS

Method 1: Calculating S, D=q/S=fp/s, and look up the density table

Method 2: An easy way -- using the chart of flow-speed curve

LOS APPLICATIONS

- Operational
  - Evaluate LOS of existing freeway segment
- Design
  - Check the adequacy of or recommend the # of lanes for a basic freeway segment

Solution

Compute free-flow speed (*use Exhibits 23-4, 23-5, 23-6, 23-7, and Equation 23-1*).

Step 1

$$FFS = BFFS - f_{LW} - f_{LC} - f_N - f_{ID}$$

$$FFS = 75 - 1.9 - 2.4 - 0.0 - 2.5$$

$$FFS = 68.2 \text{ mi/h}$$

Convert volume (veh/h) to flow rate (pc/h/ln) (*use Equation 23-2*).

Step 2

$$v_p = \frac{V}{(PHF)(N)(f_{HV})(f_p)}$$

$$v_p = \frac{2{,}000}{(0.92)(2)(f_{HV})(1.00)}$$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1) + 0}$$

$$f_{HV} = 0.930$$

$$v_p = \frac{2{,}000}{(0.92)(2)(0.930)(1.00)} = 1{,}169 \text{ pc/h/ln}$$

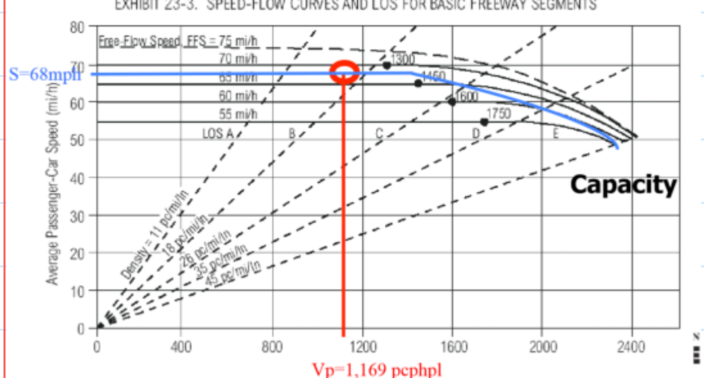
Determine level of service (*use Exhibit 23-2*).

Step 3

Solution

THE RESULTS: LOS=B

EXHIBIT 23-3. SPEED-FLOW CURVES AND LOS FOR BASIC FREEWAY SEGMENTS



Determine S & D

- For 70 < FFS ≤ 75 mph AND (3400 – 30FFS) < v_p ≤ 2400

$$S = FFS - \left[\left(FFS - \frac{160}{3}\right)\left(\frac{v_p + 30FFS - 3400}{30FFS - 1000}\right)^{2.6}\right]$$

- For 55 < FFS ≤ 70 mph AND (3400 – 30FFS) < v_p ≤ (1700 + 10FFS)

$$S = FFS - \left[\frac{1}{9}(7FFS - 340)\left(\frac{v_p + 30FFS - 3400}{40FFS - 1700}\right)^{2.6}\right]$$

- For 55 < FFS ≤ 75 mph AND v_p < (3400 – 30FFS)

$$\boxed{S = FFS}$$

For Vp = 1,169 pc/h/ln and FFS = 68 mi/h,

- S=FFS=68mi/h;
- D=Vp/S=(1169pc/h/ln)/(68mi/h)=17pc/h/ln

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Example II – Design Problem

The Freeway New suburban freeway is being designed.

The Question How many lanes are needed to provide LOS D during the peak hour?

The Facts

- N √ 4,000 veh/h (one direction),
- f_{HV} √ Level terrain, E_T, E_R ⇒ Table
- P_T=15% √ 15 percent trucks,
- LW √ 12-ft lane width, ⇒ $f_{LW}=0$
- √ 0.85 PHF,
- ID √ 1.50 interchanges per mile,
- P_R= 3% √ 3 percent RVs, and
- Lc √ 6-ft lateral clearance. ⇒ $f_{LC}=0$

$f_p = 1.00$ commuter traffic

BFFS=70 MPH (urban/suburban)

BFFS=75 MPH rural

Outline of Solution:

All input parameters are known except # of lanes. Use "Trial and error" to calculate Vp and FFS starting with a 4-lane freeway and then increase the number of lanes to 6,8 and so forth until LOS d is achieved.

Solution

Freeway Capacity
2,400 pcphpl (70 mph)
2,350 pcphpl (65 mph)
2,300 pcphpl (60 mph)
2,250 pcphpl (55 mph)

Step 1: First try the two-way 4-lane option

Convert volume (veh/h) to flow rate (pc/h/ln) (*use Equation 23-2*).

$$v_p = \frac{V}{(PHF)(N)(f_{HV})(f_p)}$$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + (0.15)(1.5 - 1) + 0.03(1.2 - 1)}$$

$$f_{HV} = 0.925$$

$$v_p = \frac{4{,}000}{(0.85)(2)(0.925)(1.00)} = 2{,}544 \text{ pc/h/ln} > 2400 \text{ pc/h/ln}$$

Step 2: Check if v_p exceeds the capacity or not

The four-lane option is not acceptable since 2544 pc/h/ln exceeds capacity of 2400 pc/h/ln. ✕

Step 3: Try the 6-lane option

$$v_p = \frac{4{,}000}{(0.85)(3)(0.925)(1.00)} = 1{,}696 \text{ pc/h/ln} < 2400$$ ✓

Step 4: Compute free-flow speed for a six-lane freeway (*use Exhibits 23-4, 23-5, 23-6, 23-7, and Equation 23-1*).

$$FFS = BFFS - f_{LW} - f_{LC} - f_N - f_{ID}$$

$$FFS = 70 - 0.0 - 0.0 - 3.0 - 5.0$$

$$FFS = 62.0 \text{ mi/h}$$

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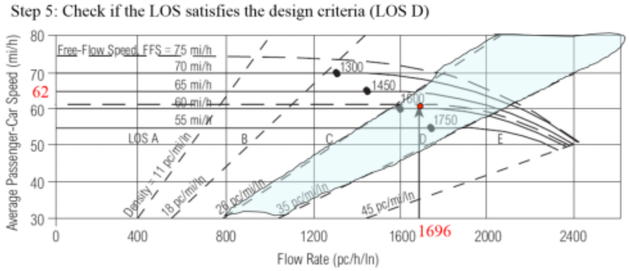
Solution (cont'd)

Step 5: Check if the LOS satisfies the design criteria (LOS D)



5-1: Interpolate the speed curve using S=62mph;

5-2: Draw a vertical line at Vp=1696, and find the intersection point with speed curve of S=62;

5-3: Determine LOS

THE RESULTS:

6 lanes needed (LOS=D)

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Determine S & D

- For 70 < FFS ≤ 75 mph AND (3400 – 30FFS) < v_p ≤ 2400
  $$S = FFS - \left[\left(FFS - \frac{160}{3}\right)\left(\frac{v_p + 30FFS - 3400}{30FFS - 1000}\right)^{2.6}\right]$$
- For 55 < FFS ≤ 70 mph AND (3400 – 30FFS) < v_p ≤ (1700 + 10FFS)
  $$S = FFS - \left[\frac{1}{9}(7FFS - 340)\left(\frac{v_p + 30FFS - 3400}{40FFS - 1700}\right)^{2.6}\right]$$
- For 55 < FFS ≤ 75 mph AND v_p < (3400 – 30FFS)
  $$S = FFS$$

For Vp = 1,696 pc/h/ln and FFS = 62 mi/h,

- (3400-30FFS)=(3400-30*62)=1540pc/h/ln ;
- 1700+10FFS=1700+10*62=2320pc/h/ln
- Using Equation 2, S= 61.8mi/h
- D=Vp/S=(1696pc/h/ln)/(61.8mi/h)=27.4pc/h/ln

Week 5

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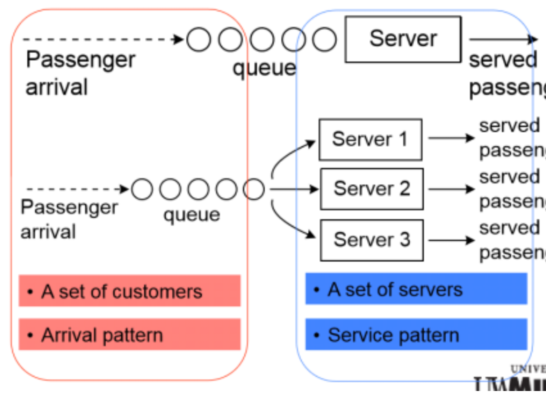
Highway Capacity: Queuing Systems

Queuing Systems
- Airport screening
- Work zone
- Toll Plazas
- Signalized Intersections

A Queueing Process & A Queueing System

Arrive → Wait in line → Receive Service → Leave



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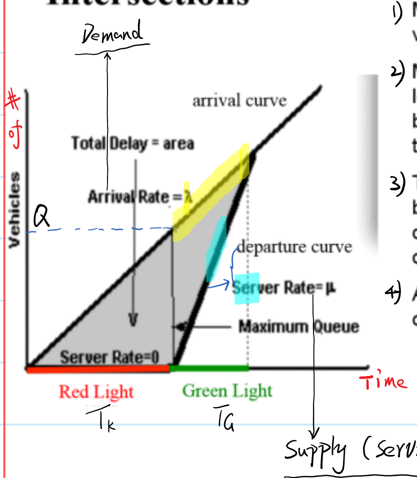
Queueing Diagram -- Signalized Intersections



1) Max Queue = the longest vertical line = q

2) Max time in queue = the longest horizontal line between the arrival and the departure line = T_R

3) Total delay = The area (Veh × Time) between the departure curve and the arrival curve $\sum_{\text{All veh}}$ the delay of each vehicle

4) Ave. delay = Total delay/# of vehicles in queue

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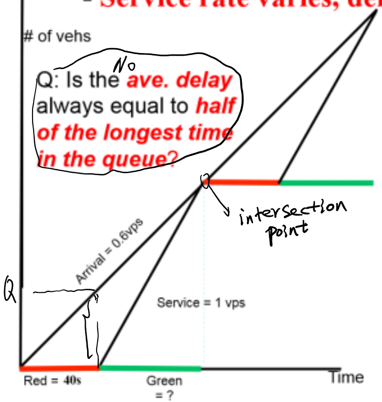
Typical Problems

- Service rate varies, demand constant
- Demand varies, service rate constant

Example 1: Traffic Signal Application
- Service rate varies, demand constant



1. What is the minimum green time to ensure 100% queue cleared?
2. Will this intersection spill back? No
3. What is the longest time any single vehicle was in the queue? 40s
4. What is the max queue? $Q = 40 \times 0.6 = 24$ veh
5. What is the total delay?
6. What is the average delay?

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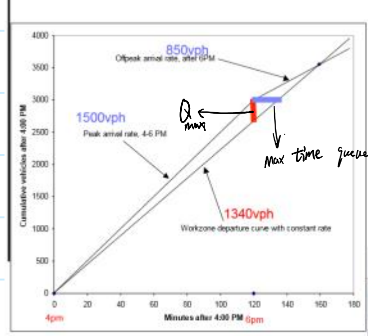
Example 2: Work Zone☆
-Demand varies, service rate constant



— Demand

- V_P Traffic flow rate is 1500vph from 4-6pm (peak), and 850vph after 6pm (off-peak)
- Single lane capacity is 1340vph — supply

1) $Y_{arrival} = \lambda + T_{arrival}$ $\quad Y_{service} = \mu \times X_{service} = 0 \times T_R + 1 \times T_G$

$= \lambda \times (T_R + T_G)$

$T_G = 60s$

2) if $\lambda < \mu$, Enough T_G, no spill back; if $\lambda > \mu$, spill back

5) obtuse triangle. $\Delta = \frac{1}{2} \times 40 \times (40 \times 60) \times 0.6 = 1200$ veh·s / 6) Ave Delay $= \frac{\Delta}{\#} = \frac{1200}{60} = 20$ s

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Queue Model Description

- x/y/z – X arrival patterns – Y service patterns – Z number of servers
  (Demand above "arrival", Supply above "service")
- D – deterministic → Constant, unifrom
- M – Markovian (Poisson arrival) → ramdomly
- G – General distribution (other than D & M)

Examples: Queue Model Description

- x/y/z:
  - M/M/1:
    - Poisson arrival, Poisson service time, 1 server
  - M/M/z:
    - Poisson arrival, Poisson service time, z servers
  - M/D/1
    - Poisson arrival, constant service time, 1 server
  - M/G/1
    - Poisson arrival, any service time where Poisson is not appropriate, 1 server

Toll Plaza

Airport screening

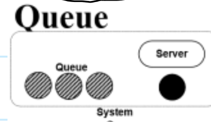
All way stop

Ramp meters

Parameters we are concerned

Average Arrival Rate	λ
Average Service Rate	μ
Average Waiting Time in Queue	$\overline{W}$
Average Time in System	$\bar{t}$ *(Waiting time + serving time)*
Average Length of Queue	$\overline{Q}$
Average # of customers in system	$\bar{n}$ *(Waiting # + serving #)*
Probability of n customers in system	p_n
Utilization ratio	$\rho = \frac{\lambda}{\mu}$ or $\rho = \frac{\lambda}{z\mu}$

M/M/1, Single Server, Single Queue



- x : Poisson arrival:
- y : Poisson service:
- z: 1 server.

$$\overline{Q} = \frac{\rho^2}{1-\rho}$$

$$\overline{W} = \frac{\lambda}{\mu(\mu-\lambda)} = \frac{\rho}{\mu-\lambda}$$

$$\bar{t} = \frac{1}{\mu-\lambda} \qquad \bar{n} = \frac{\lambda}{\mu-\lambda}$$

$$p_n = (1-\rho)\rho^n$$

$\rho < 1$

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Example M/M/1, Airport Security Screening

- Average arrival rate = 2 passengers per minute (PPm)
- Average server rate = 3 passengers per minute

*: Utilization ratio, $\rho = 2/3 < 1$ (ok)

Computations

Average # of customers in system $\quad \bar{n} = \frac{\lambda}{\mu-\lambda} = \frac{2}{1} = 2$ persons

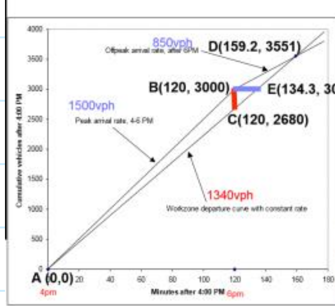
Average Length of Queue $\quad \overline{Q} = \frac{\rho^2}{1-\rho} = \frac{(2/3)^2}{1-2/3} = 1.333$ persons

Average Waiting Time in Queue $\quad \overline{W} = \frac{\lambda}{\mu(\mu-\lambda)} = \frac{2}{3(3-2)} = 0.667$ minute

Average Time in System $\quad \bar{t} = \frac{1}{(\mu-\lambda)} = \frac{1}{(3-2)} = 1$ minute

Example 2: Work Zone ★ for Graduate

-Demand varies, service rate constant



- Traffic flow rate is 1500vph from 4-6pm (peak), and 850vph after 6pm (off-peak)
- Single lane capacity is 1340vph

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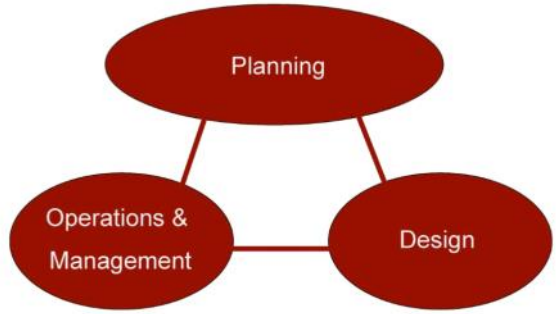
What is Transportation Planning?

-*Planning* is the process of deciding what to do and how to do it.
- Short-term vs. Long-term

-Transportation planning
- should allow transportation, land use, economic development and social planning decisions to be coordinated.

-The goal: create a better, more prosperous and more sustainable place.
- ***Better transportation, better city***

Emphasis of Areas (Course Overview)



Some Pitfalls to avoid in Transportation Planning

1. Confusion between development and growth
2. Overlooking the broader impacts of transportation plans
3. Focusing on mobility and not accessibility

Smart Growth
||
growth ⟶ bigger (quality) Development ⟶ Better (quality)

predicted or induced.

1. Plan for Development, not for Growth

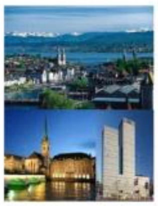
Planners must make a distinction between growth (increased quantity) and development (increased quality).

In other words, growth means getting bigger, while development means getting better - *Plan for better, not for bigger*

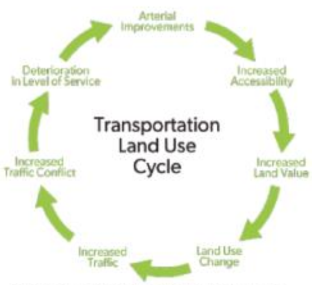
Transportation plans should contribute to the development of a place not simply to its growth.

Zurich - The Little Big City (Area:87.88 km²/33.93mi²; Population: 390,474; Density: 4,400km²/12,000mi²)

In Zurich in the 1970s, transportation planning was done with the explicit intent that they wonted to develop but not necessarily to grow. Other places might be willing to accommodate both development and growth. But the important point is that development should be given the higher priority - not growth.



Transportation and land use cycle



Transportation and land use: Which is the chicken and which is the egg?

2. Don't Overlook the Wider Impacts of Transportation Plans

Transportation plans always have wide ranging impacts, affecting not just travel but also economic, social and environmental aspects of our lives.

These impacts may be short term or much longer term, and they may extend across geographic and political boundaries.

If we don't consider these wider impacts, our plans will lead to unintended or undesirable consequences.

*First level - **Direct impacts** or changes in travel conditions and costs.*

*Second level - **Current indirect impacts** or changes in travel behavior, tax revenue, and external impacts.*

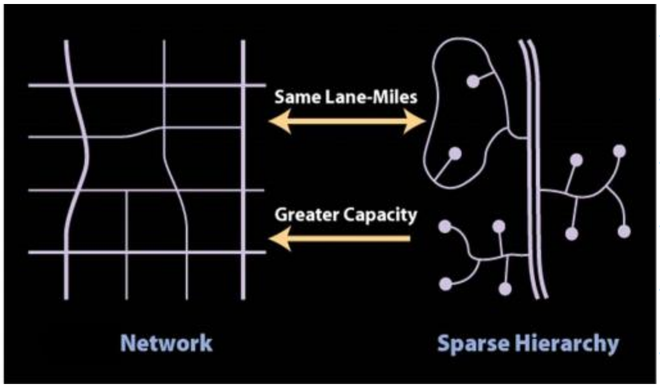
*Third level - **Long-term indirect impacts** or changes in land use and economic development*

A example of Road widening:

can have the first-level impacts of initially reducing traffic congestion and increasing vehicle traffic speeds.
A second-level impact is that the increased traffic capacity may attract additional travel from other routes and times (Rebound Effects).
A third-level impact may be that over the long run, land use patterns become more dispersed and automobile dependent (Land Use Impacts). This is one source of so called 'induced traffic' – traffic over and above what one would expect from just extrapolating from the past rate of growth.

3. Focus on Accessibility not Mobility

Traffic
Conventional transportation often reflects the assumption that transportation means motor vehicle traffic.
Mobility
A more comprehensive approach reflects the assumption that transportation means personal mobility, measured in terms of person-trips and person-kilometers.
Access
The most comprehensive definition of transportation is ***Accessibility****, the ability to reach desired goods, services and activities. This is the ultimate goal of transportation, and so is the best definition to use in transportation planning.*



Acessibility
Mobility

Basic elements of transportation planning

Problem definition	The key transportation/economic/land use issues, study area, the scope of the study, and establishment of a planning committee
Define goals, objectives and criteria	Define objectives (e.g., Reduce traffic congestion), Establish criteria (e.g., Travel time, delays), Define constraints, Establish design standards
Data collection	Traffic data, census info, interviews of households about their travel patterns, land use/development trends/environmental/financial resources
Forecasts	Forecasts of future travel, which requires forecasts of future population, land use and economic conditions, and understanding of how people make travel choices
Development of alternatives	Consider options of future land use and transportation systems
Evaluation of alternatives	Forecasts are used to determine and compare the performance of alternatives in meeting goals, objectives and criteria
Implementation plan	Once an alternative is chosen, design necessary elements of the facility and implementation plans

Week6

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Transporatation planning

1、 4 step Travel forecasting

① Trip Generation   land use patterns
{ Production P
{ Attraction A

② Trip Distribution   PA matrix

Zone1

A \ P	①	②	③	④
zone2 ①				
②				
③				
④				

③ Mode choice   split into diffirent modes

④ Trip assignment

2、 Goals of Travel Forecasting

3、 Measures of Efectiveness

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① Trip Generation

origin & Destination (can use for one person).
P & A (for zones)

② Land use and Trip Making

Trip Generation Manual

Dwelling unit

③ TAZ: Traffic Aanalysis Zone.

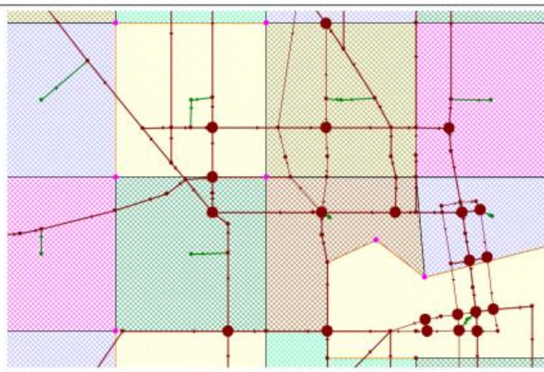
Trip purpose with a TAZ
HBW
HBNW
NHB
special
Trough Trips
Truck/Taxi

Cross classification

Trip Purposes within a TAZ

Home-Based Work (HBW)
Home-Based Nonwork (HBNW) (HBO)
Nonhome-Based (NHB)
Special
Through Trips
Truck/Taxi

Traffic Analysis Zones



For large-area analysis, site-by-site analysis is tedious

Regression Models for Trip Generation within a TAZ

- Regression
- a’s are rates and x’s are levels of activity

$$A = a_0 + a_1 x_1 + a_2 x_2 + a_3 x_3 \cdots$$

Production and Attraction

- Production: Regression or Cross-classification
- Attraction: Regression
- Obtained independently

Balancing

- Total P is not equal to total A, why?
- Need to proportionally increase As (or Ps) to assure that:

$$\sum_i P_i = \sum_i A_i$$

Example: NCHRP Report #365

- HBW A’s = 1.46*TotalEmpl
- HBNW A’s = 9*Retail + 0.5*Nonretail + 0.9*DUs + 1.7*Service + 2*CBDRetail
- NHB = 4.1*Retail + 0.5*Nonretail + 0.5*DUs + 1.2*Service + 2*CBDRetail

Pros and Cons?

Convenient but neglect the variation between households within the TAZ

An Alternative Way

- Cross Classification – put households into groups based on size, income, and auto availability

People in Household

Autos		1	2	3	4	5+
	Zero	2.6	4.8	7.4	9.2	11.2
	One	4.0	6.7	9.2	11.5	13.7
	Two	4.0	8.1	10.6	13.5	16.7
	Three	4.0	8.4	11.9	15.1	18.0

Example:

Trip rates

	2	3	4
One	6.7	9.2	11.5
Two	8.1	10.6	13.5

Demographic Forecast

	2	3	4
One	30	40	25
Two	60	80	50

- What is the total trip production?
- P_{total} = 6.7*30 + 9.2*40 + 11.5*25 + 8.1*60 + 10.6*80 + 13.5*50 = 2865.5

Example

- How to balance P & A?

TAZ i	Production i	Attraction i
1	793	2527
2	1143	3627
3	5803	1757
4	6583	1497
Total	14322	9408

4 -- Trip distribution

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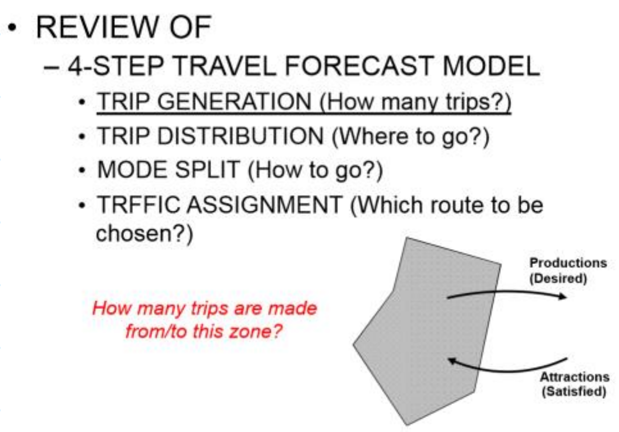
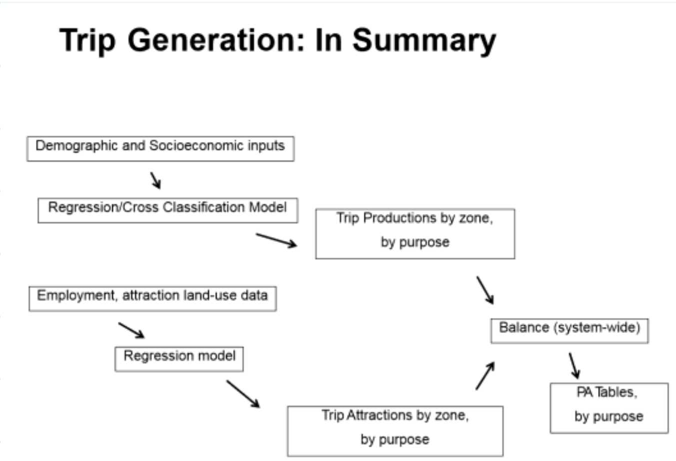
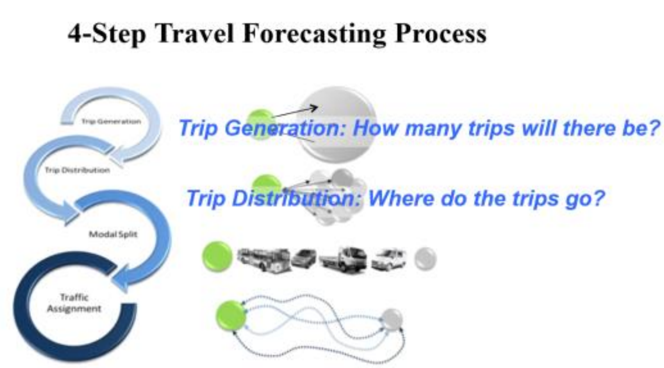
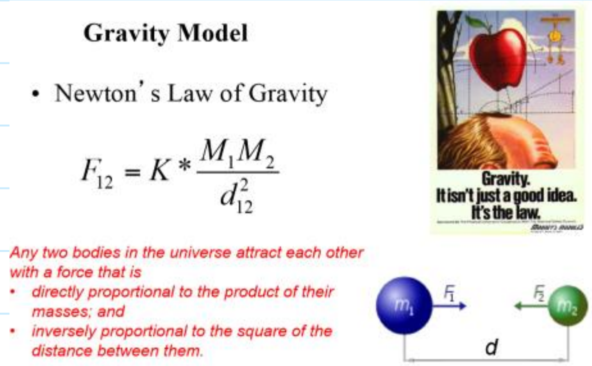








$T_{31} = K\frac{P_3A_1}{t_{31}^2}$ $T_{32} = K\frac{P_3A_2}{t_{32}^2}$ $T_{34} = K\frac{P_3A_4}{t_{34}^2}$ $T_{35} = K\frac{P_3A_5}{t_{35}^2}$

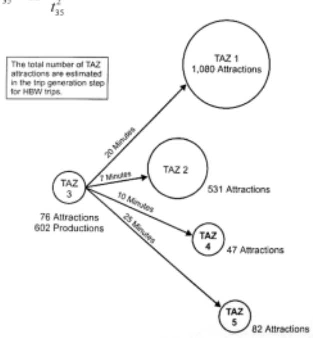
K=?

$$T_{31}+T_{32}+T_{34}+T_{35}=P_3$$

$$K\frac{\cancel{P_3}A_1}{t_{31}^2}+K\frac{\cancel{P_3}A_2}{t_{32}^2}+\frac{\cancel{P_3}A_4}{t_{34}^2}+\frac{\cancel{P_3}A_5}{t_{35}^2}=\cancel{P_3}\ 1$$

$$K=\frac{1}{\sum_{j=1}^{n} A_j / t_{ij}^2};\quad \text{let } F_{ij}=\frac{1}{t_{ij}^2}$$

$$K=\frac{1}{\sum_{j=1}^{n} A_j F_{ij}}$$



TAZ "Atractiveness"

$$P_3 = K \times P_3 \times \left(\frac{A_1}{t_{31}^2} + \frac{A_2}{t_{32}^2} + \frac{A_4}{t_{34}^2} + \frac{A_5}{t_{35}^2}\right)$$

$$k = \frac{1}{\left(\frac{A_1}{t_{31}^2} + \frac{A_2}{t_{32}^2} + \frac{A_4}{t_{34}^2} + \frac{A_5}{t_{35}^2}\right)}$$

$$K = \frac{1}{\sum_{j=1}^{5} \frac{A_j}{t_{ij}^2}}$$

let $F_{ij} = \frac{1}{t_{ij}^2}$

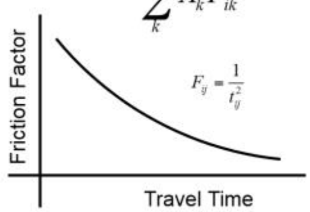
F_{ij} friction factor

Gravity Model for trip distribution

- An analogue of Newton's law of gravity
- Trips from zone i to zone j:

$$T_{ij} = P_i \frac{A_j F_{ij}}{\sum_j A_j F_{ij}}$$

$$\sum_k A_k F_{ik}$$



P_i is the production of zone i;
A_j is the attraction of zone j;
F_{ij} is the friction factor between zones i and j.

"A measure of proximity between two zones"

$$t_{ij} = P_i \times \frac{A_j F_{ij}}{\sum_j A_j F_{ij}} \rightarrow F_{ij} = \frac{1}{t_{ij}^2}$$

j=1; i=A

ex: $t_{A1} = P_A \times \frac{A_1 F_{A1}}{A_1 F_{A1} + A_2 F_{A2} + A_3 F_{A3} + A_4 F_{A4}}$

C — computed attraction

A, Given.

Note: To produce a doubly constrained
gravity model, repeat the trip distribution
computations using modified attraction
values
so that the numbers attracted will be
increased or reduced as required.

Adjusted attraction, $A_{jk} = \frac{A_j}{C_{j(k-1)}} A_{j(k-1)}$

② A

✓ t_{ij} $\hat{F}_{ij}$ → more accurate.

Single constraint.

① $\hat{A}$ → more accurate

t_{ij} F_{ij}

Double constraint

A *singly constrained gravity model* requires that **computed and actual productions** must be equal, whereas
a *doubly constrained gravity model* requires that **computed and actual productions and attractions** must be equal.

The singly constrained gravity model may be preferred if the **friction factors** are more reliable than the attraction values.
The doubly constrained gravity model is appropriate if the **attraction values** are more reliable than friction factors. To illustrate either choice, consider the following example.

6

2016年10月22日 22:52

12.3.1 The Gravity Model

$$T_{ij} = P_i \left[\frac{A_j F_{ij} K_{ij}}{\sum_j A_j F_{ij} K_{ij}} \right] \tag{12.3}$$

where

T_{ij} = number of trips that are produced in zone i and attracted to zone j
P_i = total number of trips produced in zone i
A_j = number of trips attracted to zone j
F_{ij} = a value which is an inverse function of travel time
K_{ij} = socioeconomic adjustment factor for interchange ij

To create a doubly constrained gravity model where the computed attractions equal the given attractions, calculate the adjusted attraction factors according to the formula

$$A_{jk} = \frac{A_j}{C_{j(k-1)}} A_{j(k-1)} \tag{12.4}$$

where

A_{jk} = adjusted attraction factor for attraction zone (column) j, iteration k
A_{jk} = A_j when $k = 1$
C_{jk} = actual attraction (column) total for zone j, iteration k
A_j = desired attraction total for attraction zone (column) j
j = attraction zone number, $j = 1, 2, \ldots, n$
n = number of zones
k = iteration number, $k = 1, 2, \ldots, m$
m = number of iterations

7 --modal split

2016年10月19日 23:54

1. Trip generation (P_i, A_j)

2. Trip distribution (T_{ij})

3. Mode Split

What affects people's mode choice?

Characteristics of the trip: trip distance, time of day, trip purpose

Characteristics of the trip maker: Income, # of autos available, family size, residential density, gender

Characteristics of the transportation system: riding time, waiting time, transfers, out-of-pocket cost

Mode Split/Choice Models

Mode Choice Models are used to try to predict travelers mode choice.
Contemporary models are based on using UTILITY functions.
These functions are meant to express the level of satisfaction (for utility functions) with a given mode.
Once the utility function is calculated for each mode, the probability that a given mode will be chosen can then be calculated

Utility: the sense of pleasure, or satisfaction, that comes from consumption

Random Utility Theory I

- Mode split is a function of measured (deterministic properties) and unmeasured (random properties) of utilities.
- Utility is a measure of well being.
- V is usually a negative number

$$U = V_{deterministic} + \varepsilon$$

Random Utility Theory II

- Each traveler chooses the mode that maximizes his/her utility.

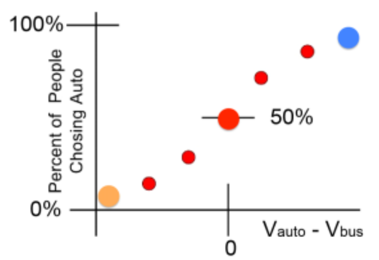
$$U_{ChosenMode} > U_{AnyOtherMode}$$

Mode Split Observations

P_{auto}=50%, P_{bus}=50%: Vauto = Vbus

P_{auto}=0%, P_{bus}=100%: Vauto >> Vbus

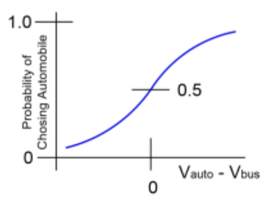
P_{auto}=0%, P_{bus}=100%: Vauto << Vbus



Logit Model, Fundamental Shape

$$P_{auto} = \frac{e^{V_{auto}}}{e^{V_{auto}} + e^{V_{bus}}} = \frac{1}{1 + e^{V_{bus} - V_{auto}}}$$

$$P_{bus} = \frac{e^{V_{bus}}}{e^{V_{auto}} + e^{V_{bus}}} == \frac{1}{1 + e^{V_{auto} - V_{bus}}}$$



Estimating Deterministic Utility

$$V_m = a_{0m} + a_1 x_{1m} + a_2 x_{2m} + a_3 x_{3m} + \dots$$

- a_{0m} is the mode specific constant
- a_1, a_2 … are empirical constants, usually negative numbers
- x_{1m}, x_{2m} … are modal properties (travel time, travel cost, etc.)

8

2016年10月19日 23:54

Example: Auto-Bus Mode Split between Finchburg and Foxville

$$V_m = a_{0m} - 0.03T_m - 0.6C_m$$

T_m is travel time in minutes

C_m is travel cost in dollars

$a_{0auto} = 0$; $a_{0bus} = -0.2$

Finchburg to Foxville Trip Data

- 300 people make the trip daily
- Travel time by auto is 15 min; travel time by bus is 35 min.
- Out-of-pocket costs by auto is \$0.80; transit fare is \$0.50
- How many travelers take transit?

	a_{0m}	T_m	C_m
Auto	0	15	\$ 0.8
Bus	-0.2	35	\$ 0.5

$T_{ij} = 3000$ people

$$P_{auto} = \frac{e^{V_{auto}}}{e^{V_{auto} + V_{bus}}} = .65\%$$

$$V_{auto} = 0 - 0.03 \times 15 - 0.6 \times 0.8 = -0.93$$

$$V_{bus} = -0.2 - 0.03 \times 35 - 0.6 \times 0.5 = -1.55$$

$$P_{bus} = \frac{e^{-1.55}}{e^{-0.93} + e^{-1.55}} = 35\%$$

$$\# \text{ of traveller} = T_{ij} \times P_{bus} = 300 \times 35\% = 105$$

```
for(rr=0; rr< RECYCLING_PLANTS; rr++)
{
```

12.4.2 Logit Models

An alternative approach used in transportation demand analysis is to consider the relative utility of each mode as a summation of each modal attribute. Then the choice of a mode is expressed as a probability distribution. For example, assume that the utility of each mode is

$$U_x = \sum_{i=1}^{n} a_i X_i \tag{12.8}$$

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where

U_x = utility of mode x
n = number of attributes
X_i = attribute value (time, cost, and so forth)
a_i = coefficient value for attributes i (negative, since the values are disutilities)

If two modes, auto *(A)* and transit *(T)*, are being considered, the probability of selecting the auto mode *A* can be written as

$$P(A) = \frac{e^{U_A}}{e^{U_A} + e^{U_T}} \tag{12.9}$$

This form is called the *logit model,* as illustrated in Figure 12.10, and provides a convenient way to compute mode choice. Choice models are utilized within the urban transportation planning process and in transit marketing studies.

Value of Travel Time

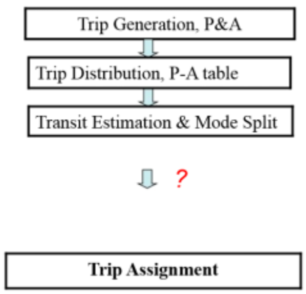
$$V_m = a_{0m} - 0.03T_m - 0.6C_m$$

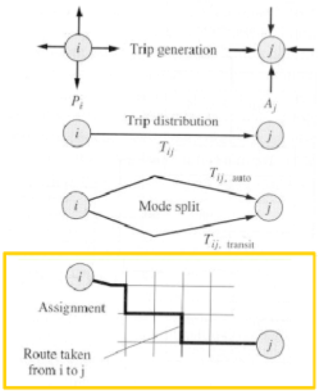
- There are -0.03 utils per minute
- There are -0.6 utils per dollar

VoT = -0.03/-0.6 = \$0.05 per minute

9 -- Traffic Assignment(Travel Demand)

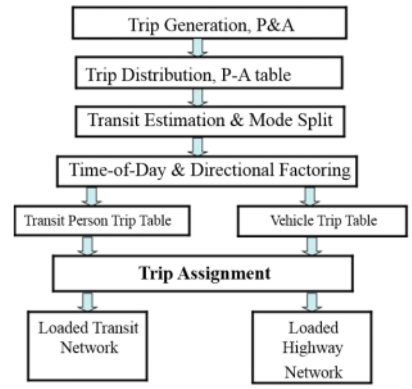
Traffic Assignment

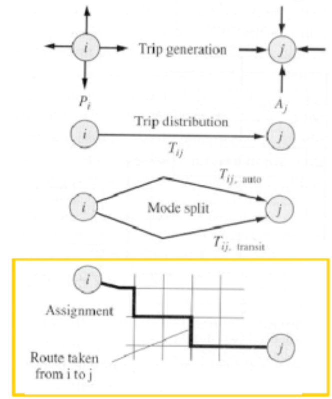




Which route will be taken?

Traffic Assignment





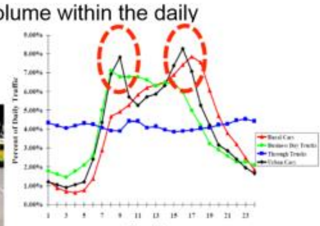
Hourly Volume

not AADT

- Hourly volume
- Why we need it?
- DDHV - directional design hourly volume

  DDHV = AADT (or ADT) * K * D

K - proportion of peak hour volume within the daily volume （urban: 8-13%）
D - directional factor



Person Trips vs. Vehicle Trips Example

Depends

Usually adjusted by average auto occupancy

Example:

If:

- average auto occupancy = 1.2
- number of person trips from zone 1 to zone 2= 550

So:

Vehicle trips from zone 1 to zone 2

= 550 person trips/1.2 persons per vehicle = **458.33 vehicle trips**

Traffic Assignment Techniques

- Minimum-path techniques (AON)
- With Capacity constraints
- User Equilibrium (UE)
- System Optimal (SO)

$t_1 < t_2$ $q \longrightarrow$ Route 1

$$\begin{cases} q_1 = 6000; \\ q_2 = 0 \end{cases}$$

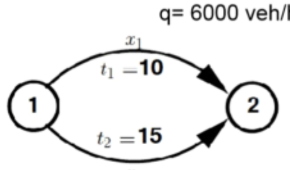
①

Minimum-path techniques (+ “all-or-nothing” trip loading)

Assumption: Travelers want to use the minimum impedance route between two points.

Given impedance values, assignment algorithms find minimum paths (or shortest paths) to get from point A to all other locations (to which trips are distributed).

q= 6000 veh/h



Solution: X1= 6000
X2=0

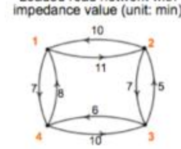
In this method, all trips between a given origin and destination are loaded on the links comprising the minimum path and nothing is loaded on the other links (è No consideration of the link capacities.)

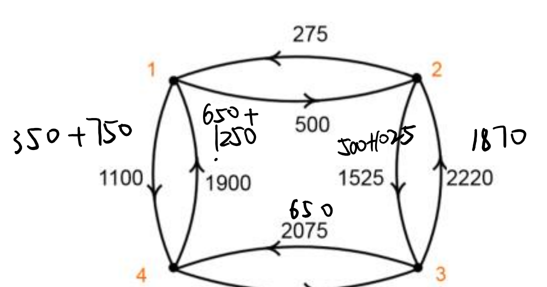
Example of AON Assignment

Vehicle trip table (unit: veh/h)

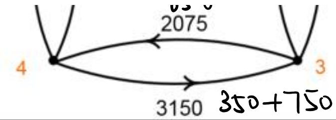
		Destination 1	2	3	4
Origin	1	0	500	750	350
	2	275	0	1050	475
	3	650	1870	0	950
	4	1250	350	2050	0

Loaded road network with impedance value (unit: min)





loaded on the links comprising the minimum path and nothing is loaded on the other links. (è No consideration of the link capacities.)



350+750+2050

② UE: Min Personal Travel Time

$T_R = T_G$

Theory of User Equilibrium

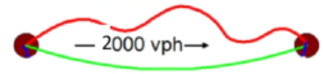
Travelers will select a route so as to minimize their personal travel time between their origin and destination. User equilibrium (UE) is said to exist when travelers at the individual level cannot unilaterally improve their travel times by changing routes.

Wadrop definition: *A.K.A. Wardrop's 1st principle*
"The travel time between a specified origin & destination on all used routes is equal, and less than or equal to the travel time that would be experienced by a traveler on any unused route"

Link performance function LPF

A Simple Example

$T_R = 3+V_R$



$T_G = 2+2V_G$

$2+2V_G = 3$

$V_G = \frac{1}{2}$

V_R and V_G are thousands of vehicles per hour. ☆

The first driver will choose the GREEN Line because its free flow travel time is 2min, compared to 3min on RED line.

The first 500 drivers will choose GREEN line as the shorter time path, but the 501st driver will find that GREEN line is no longer the shortest time path.

Any subsequent driver will choose the path that has the shortest travel time for the flow levels that prevail at the time of the decision.

$T_R + T_G \Rightarrow \begin{cases} 3+V_R = 2+2V_G \\ V_R + V_G = 2 \end{cases} \Rightarrow \begin{cases} V_R = 1 \\ V_G = 1 \end{cases}$

Basic Assumptions

1. Travelers select routes on the basis of route travel times only
   - People select the path with the shortest TT
   - Premise: TT is the major criterion, quality factors such as "scenery" do not count
   - Generally, this is reasonable
2. Travelers know travel times on all available routes between their origin and destination
   - Strong assumption: Travelers may not use all available routes, and may base TTs on perception
   - Some studies say perception bias is small

With capacity restraints

General BPR Curve

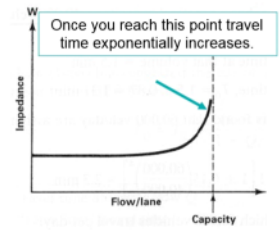
$$t = t_0 \left[1 + \alpha \left(\frac{v}{c}\right)^{\beta} \right]$$

Traditional BPR Curve

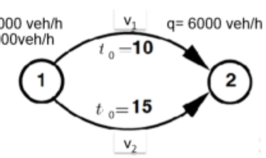
$$t = t_0 \left[1 + 0.15 \left(\frac{v}{c}\right)^{4} \right]$$

t = travel time; t_0 = free travel time;
v = volume; c = practical capacity
α and β are parameters that depend on functional class

0.15 4



c1= 3000 veh/h
c2=2000veh/h
q= 6000 veh/h



$T_1 = T_2$

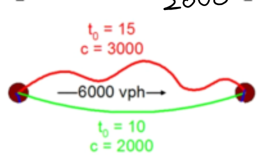
Algebra Solution Method

- $t_{red} = t_{green}$

$$t_{0red} \left[1 + 0.15 \left(\frac{v_{red}}{c_{red}}\right)^{4} \right] = t_{0green} \left[1 + 0.15 \left(\frac{v_{green}}{c_{green}}\right)^{4} \right]$$

3000   2000

- $V_{red} + V_{green} = 6000;$

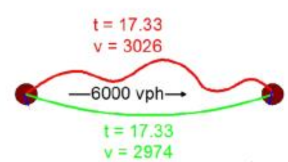


Traditional BPR Function

$$t = t_0 \left[1 + 0.15 \left(\frac{v}{c}\right)^{4} \right]$$

Solution

Using Excel solver



Tool:

Data:
Goal Seek

Minizze Total system travel Time

3 Theory of System-Optimal Route Choice

Wardrop's Second Principle:
Preferred routes are those, which minimize total system travel time. With System-Optimal (SO) route choices, no traveler can switch to a different route without increasing total system travel time. Travelers can switch to routes decreasing their TTs but only if System-Optimal flows are maintained. Realistically, travelers will likely switch to non-System-Optimal routes to improve their own TTs.

$$\Sigma(V_r \times T_r + V_G \times T_G)$$

Formulating the SO Problem

Finding the set of flows that minimizes the following function:

$$S(x) = \sum_n x_n t_n(x_n)$$

n = Route between given O-D pair

T — $t_n(x_n)$ = travel time for a specific route

V — x_n = Flow on a specific route

$$\begin{cases} \Sigma S = V_R \times T_R + V_G \times T_G \\ \quad = V_R \times (3+V_R) + V_G \times (2+2V_G) \\ V_R + V_G = 2 \end{cases}$$

$\min S = ?$

$$\Sigma S = V_R \times (3+V_R) + (2-V_R)(2+2(2-V_R))$$
$$= 3V_R^2 - 7V_R + 12$$

$$\frac{dS}{dV_R} = 0$$

$S' = 6V_R - 7 = 0$ $\quad V_R = \frac{7}{6}$

$\Rightarrow \min S$ $\quad = 1167$ veh/hr

$V_R = 2 - V_R = \frac{5}{6}$

$= 833$ veh/hr

$T_R = 3 + \frac{7}{6} = 4.16$ min

$T_G = 2 + 2 \times \frac{5}{6} = 3.67$ min

VE: $T_G = T_R$

$\Sigma S = 8000$ veh/hr

SO:

Total system cost

$\Sigma S = 7902$ veh/hr ✓

Week 7--- highway design

horizontal
vertical
curves

Functional classification
LOS targeted
Expected traffic volume & traffic mix (mode)
Design speed
Topography
LOS targeted
Available funds
Safety
Social and environmental factors

Highway Functional Classification
- What?
- Why?
- How?
- Relationship to Design

Highway functional classification

what

What is the function of a road?

- Provides mobility (arterials)
- Provides access (locals) accessbility *
- Provides both (collectors)

why

Why classify roads?

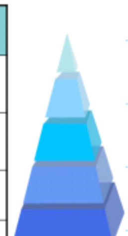
- There are approximately 4,000,000 miles of roads in the U.S.
- Some are more "important" than others

- Helps determine which level of government has responsibility
- Influences design
- Affects how they are funded
- Impacts Federal-Aid

Rural Guides

System	VMT*	Miles
Principal arterials	30-55%	2-4%
Minor arterials	45-75%	6-12%
Collectors	20-35%	20-25%
Locals	5-20%	65-75%

VMT: Vehicle miles traveled



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Highway Functional Classes (Rural Miles)

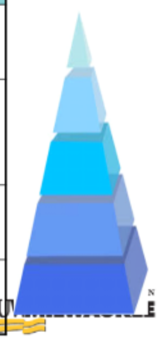
- Principal arterials 132,451 (4%)
- Minor arterials 137,875 (4%)
- Major collectors 434,090 (14%)
- Minor collectors 272,047 (9%)
- Local 2,096,837 (68%)
- Total 3,071,331

2

More traffic need more mobility

Urban Guides

System	VMT	Miles
Principal arterials	40-65%	5-10% (rural:2-4%)
Minor arterials	65-85%	15-25% (rural:6-12%)
Collectors	5-10%	5-10% (rural:20-25%)
Locals	10-30%	65-80% (rural:65-75%)



Federal-Aid
- National Highway System
  - Principal arterials
- Surface Transportation Program
  - Arterials – Urban collectors
  - Rural major collectors

How

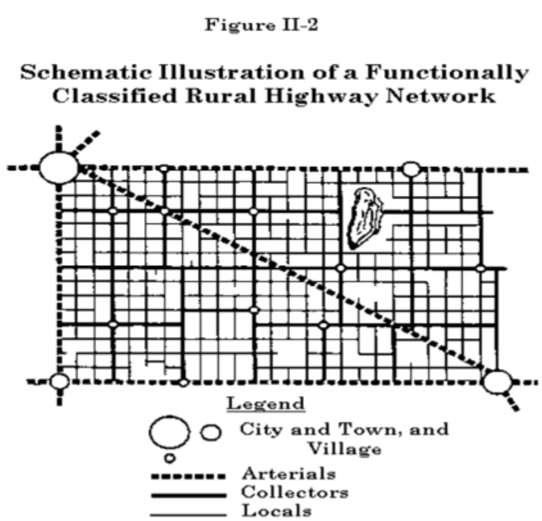
How?

- Group population centers and major travel generators
- Identify neighboring centers
- Connect the largest directly
- Connect the next group to the major centers

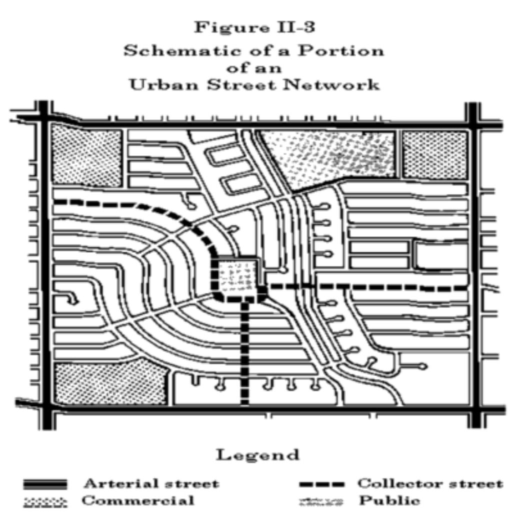
Considerations:

1. Arterials integrated network
2. density
3. Changes at urban boundaries
4. Trip length

Rural Illustration



Urban Illustration



3

Design Considerations

- AASHTO's Policy on Geometric Design of Highways and Streets 2011 (Green book)
- Arterials – promote mobility and restrict access
- Locals – promote access and limit mobility

Guidelines for selection of design LOS

(AASHTO: *A Policy on Geometric Design of Highways and Streets, 2011*)

Functional Class	Area & Terrain Type			
	Rural level	Rural rolling	Rural Mtns	Urban & Suburban
Freeway	B	B	C	C
Arterial	B	B	C	C
Collector	C	C	D	D
Local	D	D	D	D

Urban Features

- Parking
- Lighting
- Curb & Gutter
- Sidewalks
- Utilities

Access Control

- Full control
  – Freeway
- Partial control
  – Medians
  – Grade separation
  – Signal timing
  – Limit driveways and entrances

TRB
AASHTO

HCM

(AASHTO)
American Association of State Highway and Transportation Officials

(TRB)
Transportation Research Board

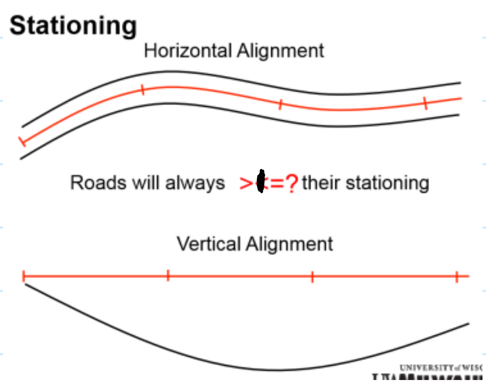
(HCM)
Highway Capacity Manual

(NCHRP)
National Cooperative Highway Research Program

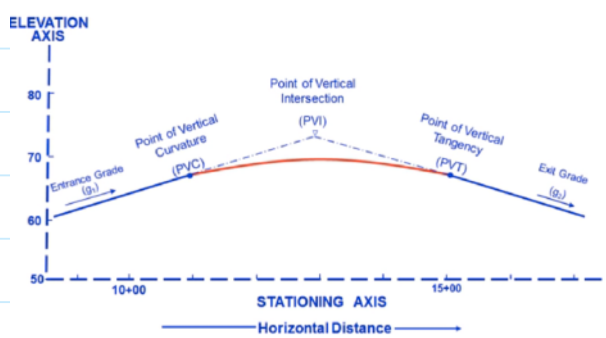
4 Geometric design ----alignment

1. Concepts
2. Vertical Alignment
a. Fundamentals
b. Crest Vertical Curves
c. Sag Vertical Curves
3. Horizontal Alignment
a. Fundamentals
b. Super-elevation

Concepts
- Alignment is a 3D problem broken down into two 2D problems
  - Horizontal Alignment (plan view)
  - Vertical Alignment (profile view)
- Stationing
  - Along horizontal alignment
  - 12+00 = 1,200 ft.



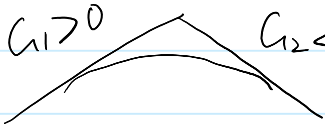
Parabolic Vertical Curve



safty issue / smothe transition

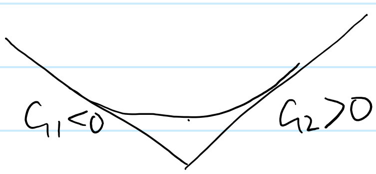
crest

$G_1 > 0$ $\quad$ $G_2 < 0$



parabolic $\underbrace{y}_{\text{elevation}} = ax^2 + b\underbrace{x}_{\text{stationing}} + c$

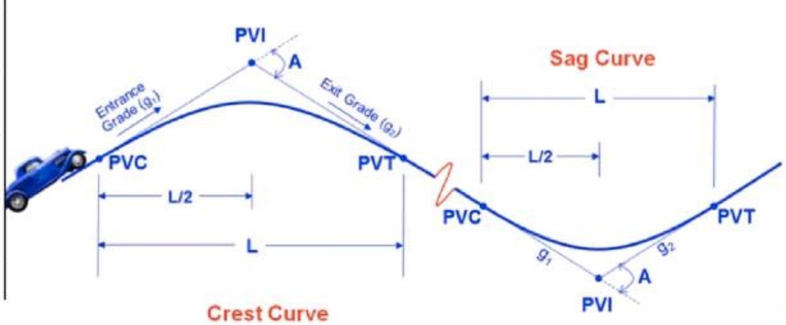
sag



$G_1 < 0$ $\quad$ $G_2 > 0$

PVC / VPC: starting point of curvature
PVI / VPI: Peak Point
PVT / VPT end point of curvature
start point of Tangency

Types of Vertical Curves



Other Terms/Relations

- Total Change in Grade, $A = G_2 - G_1$
  – Used for calculations – estimating L
- Surveys are done, typically, to the nearest 100th of a foot, so elevations and station calculations should have the same precision, e.g., 645.48 feet

5

Parabola Curve

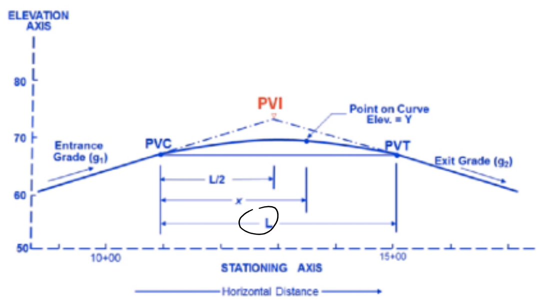
- The elevation of the vertical curve at a distance x feet from PVC is:

$Y(x) = a + bx + cx^2$

$a = Y_{PVC}$

$b = G_1$ slope

$c = \dfrac{G_2 - G_1}{2L}$



$\frac{dy}{dx} = b + 2cx \Rightarrow$ slope of tangency

$\frac{d^2y}{dx^2} = 2c \Rightarrow$ the change rate of the slope

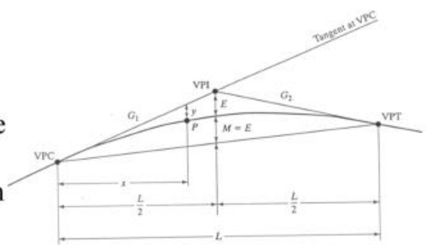
$2c = \frac{G_2 - G_1}{L}$ $c = \frac{G_2 - G_1}{2L}$

$b + 2cx = 0$

$x = \frac{b}{-2c}$

Parabola Curve

- The elevation of the vertical curve at a distance x feet from VPC is:



$Y(x) = a + bx + cx^2$ $a = Y_{VPC}$ $b = G_1$ $c = \dfrac{G_2 - G_1}{2L}$

- Tangent Elevation

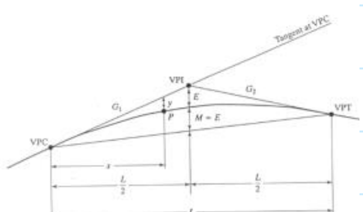
$$Y_{tan} = Y_{VPC} + G_1 x$$

- High or Low Point

$$x_{hi/lo} = L\frac{G_1}{G_1 - G_2}$$

Parabola Curve

- Offset: The distance between the curve and the extended tangent



$$offset = cx^2 = (x)^2 \frac{G_2 - G_1}{2L}$$

Offset > 0, Sag Curve

Offset < 0, Crest Curve

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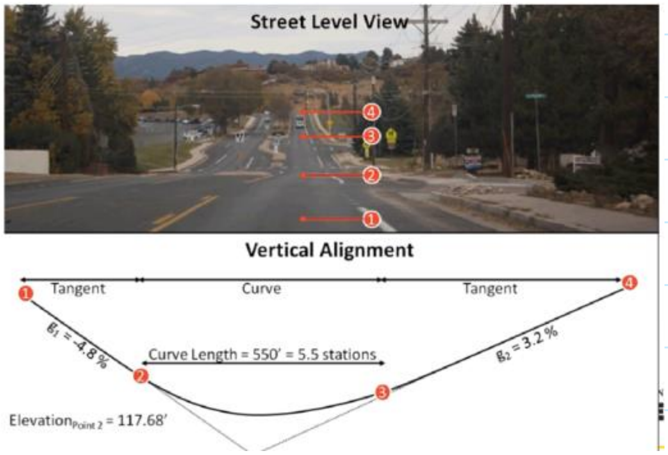
Important Notation Issue

- If G_1 and G_2 are in %, then x and L must be in 100 ft stations
- If G_1 and G_2 are in slope, then x and L must be in feet.
- If A is calculated with a sign, it will tell us whether the curve is above or below the tangent.

6

Centerline Elevation Example



$a = Y_{PVC} = 117.68$

$b = g_1 = -4.8\%$

$C = \dfrac{G_2 - G_1}{2L} = \dfrac{3.2 + 4.8}{2 \times 5.5} = \dfrac{8}{11} \times 10^{-4}$

$X_{high/low} = \dfrac{G_1}{G_1 - G_2} \times L = \dfrac{-4.8}{-8} \times 5.5$

$= 0.6 \times 5.5$

$= 3.3$

Station:	46+00				4600' ↓

Week8 ---- vertical curve length

2016年11月7日 8:16

- Elevation is determined given the vertical curve length
- How to determine the vertical curve length?

Stopping Sight Distance (SSD)

$$SSD = \frac{V_1^2}{2g\left(\frac{a}{g} \pm G\right)} + V_1 \times t_r$$

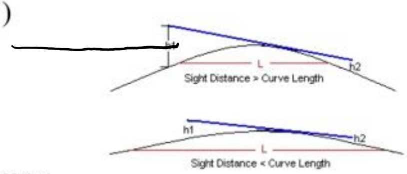
perception + stopping

- Practical stopping distance plus distance travelled during driver perception/reaction time
- Distance travelled along the roadway
- Use this to determine necessary curve length

U.S. Customary					
Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
15	55.1	21.6	76.7	80
20	73.5	38.4	111.9	115
25	91.9	60.0	151.9	155
30	110.3	86.4	196.7	200
35	128.6	117.6	246.2	250
40	147.0	153.6	300.6	305
45	165.4	194.4	359.8	360
50	183.8	240.0	423.8	425
55	202.1	290.3	492.4	495
60	220.5	345.5	566.0	570
65	238.9	405.5	644.4	645
70	257.3	470.3	727.6	730
75	275.6	539.9	815.5	820
80	294.0	614.3	908.3	910

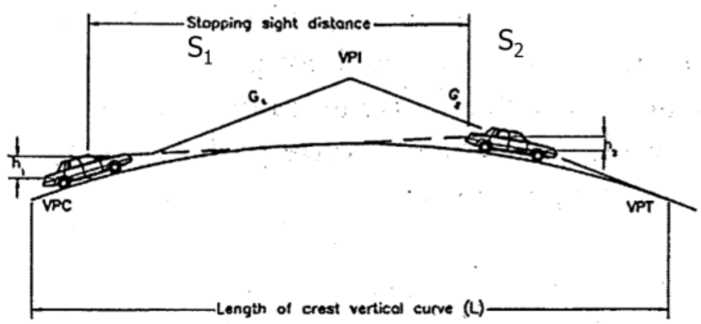
Vertical Curve AASHTO Controls (Crest)

- Minimum length must provide stopping sight distance (SSD) or "S" in equations
- Two situations (both assume h_1=3.5' and h_2=2.0')



Source: Transportation Engineering On-line Lab Manual, http://www.its.uidaho.edu/niatt_labmanual/

Stopping Sight Distance on Crest Curves



Minimum Length of Crest Curve

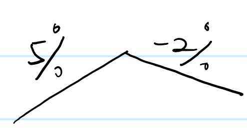
2nd step

$$L = \frac{|A|S^2}{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2} = \frac{|A|S^2}{2158} \quad S \le L$$

1st step

$$L = 2S - \frac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{|A|} = 2S - \frac{2158}{|A|} \quad S \ge L$$

Where A is the change in percent grade (A= G1-G2), h_1 is the driver eye height and h_2 is the object height.



A = 5 − (−2) = 7 ✓

A = 0.05 − (−0.02) = 0.07 ✗

2

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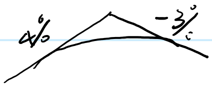
$S > L$
$L = 2S - \frac{2158}{|A|}$

Example I -- Minimum Length

- A crest curve joins an upgrade of 4% with a downgrade of 3% on a multilane highway. The speed limit is 60 mph. What should be the minimum length of the vertical curve, making the most conservative assumptions.



Speed limit 60 mph → SSD 570

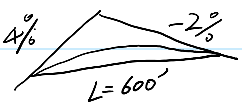
A = 7

if S ≥ L   if S ≤ L

Example II – Speed Limit

- A 600ft crest curve has a 4 percent tangent and -2 percent tangent. What speed is necessary if there is to be ample stopping sight distance?

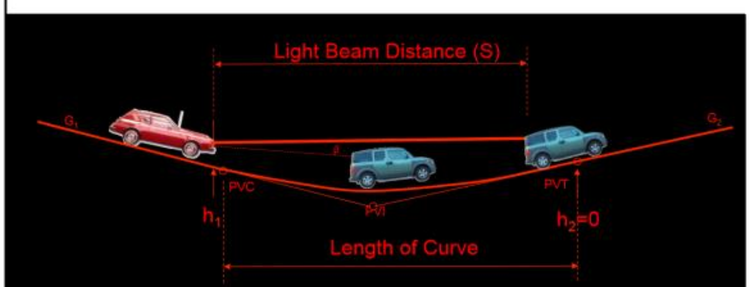
Sag Vertical Curves



A = 6

L
⇓
SSD ⇒ speed limit

Sag Vertical Curves



- Sight distance limited by headlights at night

Length Solution

- Stopping sight distance, S = 570 feet

IF $S \le L$ A S^2

$$L = \frac{|4-(-3)|570^2}{2158} = 1032\,ft$$

Since S (570ft) < L(1032ft), this equation applies.

Solution

For this curve, A=4- (-2)=6 percent and L=600ft

If S<=L, $600 = \frac{6S^2}{2158}$ $S = 465.5\,ft$

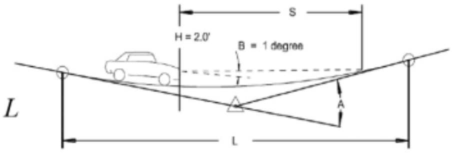
Since S (465.5ft) < L(600ft), this equation applies.

Design speed	SSD
50	425 ✓
55	495

more conservative

light beam distance

Minimum Length of Sag Curve



$$L = \frac{|A|S^2}{200(h + S\tan\beta)} \quad S \le L$$

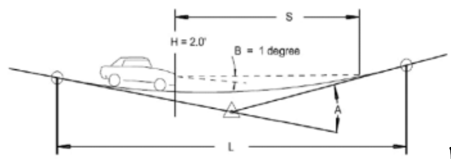
2' 1°

$$L = 2S - \frac{200(h + S\tan\beta)}{|A|} \quad S \ge L$$

Where A is the change in percent grade,
h is the height of the headlamp (2 feet)
and β is the headlight beam angle (1°).

Example I -- Minimum Length

- Downward 4%, Upward 3%, Speed Limit is 60 mph



60 mph → SSD 570 ft.

$|A| = |-7| = 7$

Sag Calculations

- Stopping sight distance, S = 570 feet
- If S<=L

$$L = \frac{|-4-3|570^2}{200(\underset{h}{2} + 570\tan \underset{\beta}{1})} = 952\,ft$$

- S (570ft) is less than L (952ft), so we are done.

Example II – Beam angle

- Fred bought a new sports car that was very sleek and low to the ground. When checking the headlamp that was only 12 inches from the road, he thought they needed to be adjusted. What should the angle of the beam be so that he can drive safely at night at 55mph?

h_2 12 inches = 1ft SSD = 495'

U.S. Customary				
Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
15	55.1	21.6	76.7	80
20	73.5	38.4	111.9	115
25	91.9	60.0	151.9	155
30	110.3	86.4	196.7	200
35	128.6	117.6	246.2	250
40	147.0	153.6	300.6	305
45	165.4	194.4	359.8	360
50	183.8	240.0	423.8	425
55	202.1	290.3	492.4	495
60	220.5	345.5	566.0	570
65	238.9	405.5	644.4	645
70	257.3	470.3	727.6	730
75	275.6	539.9	815.5	820
80	294.0	614.3	908.3	910

Solution

S<=L: $$L = \frac{|A|S^2}{200(h + S\tan\beta)} \Rightarrow \frac{|A|S^2}{200L} = h + S\tan\beta$$

S>=L: $$L = 2S - \frac{200(h + S\tan\beta)}{|A|} \Rightarrow -\frac{|A|(L-2S)}{200} = h + S\tan\beta$$

To preserve the value of the right side when h=1 instead of h=2, beam degree must be adjusted in accordance with the calculations below:

$\underset{h}{2} + 495\tan 1 = 1 + 495\tan \underset{\beta}{\beta}$ $\beta = 1.116\,\text{degrees}$

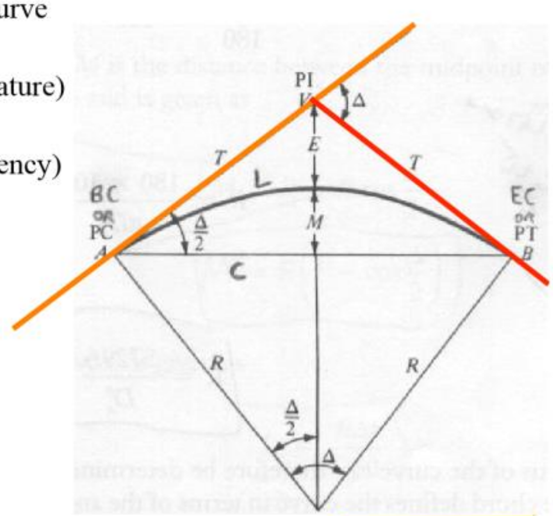
4----Horizontal Curves

Horizontal Alignment :

Tangents		Curves

Layout of a Simple Horizontal Curve

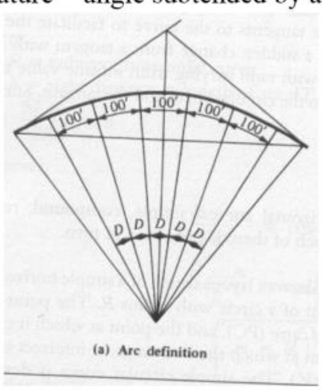
R = Radius of Circular Curve
BC = Beginning of Curve
(or PC = Point of Curvature)
EC = End of Curve
(or PT = Point of Tangency)
PI = Point of Intersection
T = Tangent Length
L = Curve Length
C = Chord Length
M = Middle Ordinate
E = External Distance
Δ = Deflection Angle



Properties of Circular Curves

Degree of Curvature

- Traditionally, the "steepness" of the curvature is defined by either the **radius (R)** or the **degree of curvature (D)**
- In highway work we use the ARC definition
- Degree of curvature = angle subtended by an arc of length 100 feet



(a) Arc definition

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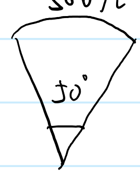
72

ex.



$D = \frac{50^\circ}{500/100} = 10^\circ$

Degree of Curvature

--Equation for D & R

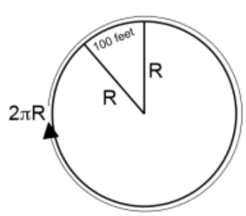
Degree of curvature = angle subtended by an arc of length 100 feet

By simple ratio: D/100 = 360/2πR

$\frac{\text{Degree}}{\text{length}}$

Therefore

R = 5729.6 / D



L

Degree of Curvature

--Equation for D & L

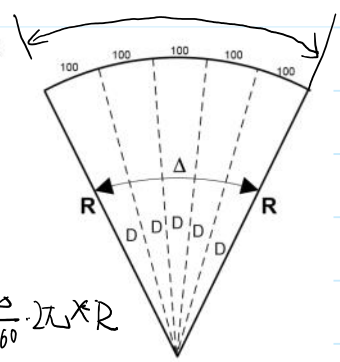
By simple ratio: D/ Δ = ?

D/ Δ = 100/L

L = 100 Δ / D

Therefore, **L = 100 Δ / D**

$L = \frac{\Delta}{360} \cdot 2\pi \times R$



Or (from R = 5729.6 / D, substitute for D = 5729.6/R)

L = Δ*R / 57.30

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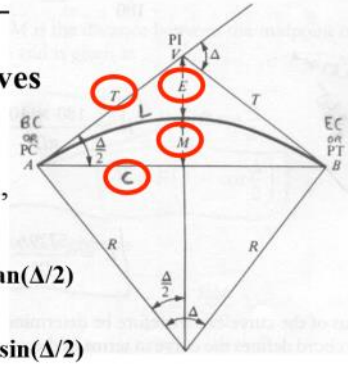
5

Properties of Circular Curves

Important Formulas…

Assuming R and Δ are given,



Tangent:	$T = R\tan(\Delta/2)$
Chord:	$C = 2R\sin(\Delta/2)$
Mid Ordinate:	$M = R - R\cos(\Delta/2)$
External Distance:	$E = R/\cos(\Delta/2) - R$

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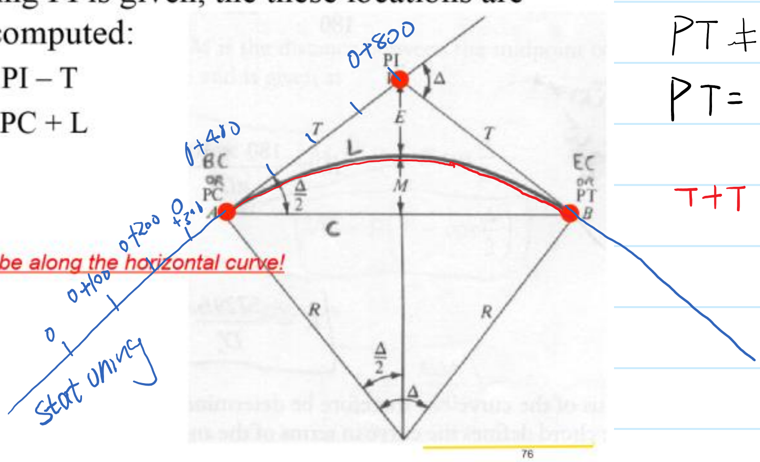
Stationing

- Assuming PI is given, the these locations are easily computed:
  - Δ PC = PI – T
  - Δ PT = PC + L



Stationing must be along the horizontal curve!

$PT \neq PI + T$

$PT = PC + L$

$T + T > L$

Example

- A 7-degree horizontal curve covers an angle of 63°15'34". Determine the radius R, the length of the curve L, and the distance M from the circle to the chord.

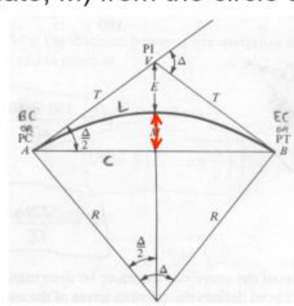
R= 5729.6/7=818.5ft

L=100*63.2594°/7=903.7ft

M=818.5* (1-cos31.6297°)=121.6ft

Discussion

- Why we need know the distance (mid ordinate, M) from the circle to the chord?



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Sight distance problem

$D = 7^\circ \qquad \Delta = 63^\circ 15' 34''$

$= 63^\circ + \frac{15}{60} + \frac{34}{3600} = 63.2594$

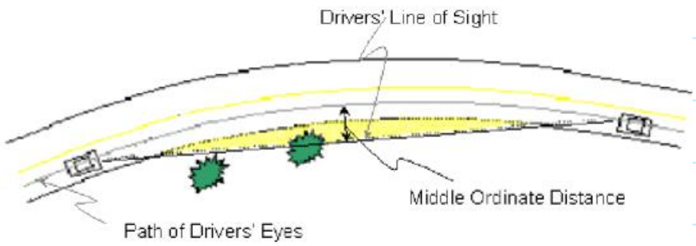
D & R $\quad R = \frac{5729.16}{D}$

D & L $\quad L = \frac{100\Delta}{D} \qquad M = R(1-\cos\frac{\Delta}{2})$

L & R $\quad L = \frac{\Delta \times R}{5730}$

6

Sight Distance on Curves



Critical Inside Lane → control the M

Δ_s $\underline{\text{in}}$ Inner most lane

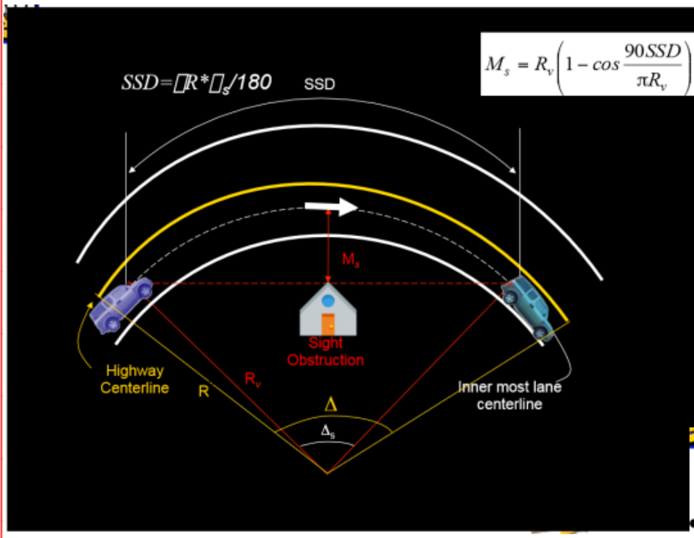
$$SSD = \frac{\pi R_v \times \Delta_s}{180}$$

$$\boxed{\frac{SSD}{\Delta_s} = \frac{2\pi R_v}{360^\circ}}$$

☆

lane width = 12 ft

$R_v = R - 6$



Sight Distance Formula

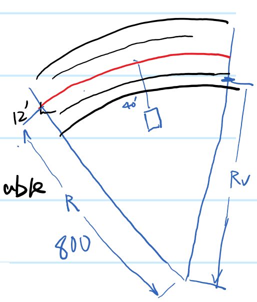
$$M_s = R_v\left(1 - \cos\frac{90SSD}{\pi R_v}\right)$$

$\frac{\Delta_s}{2}$

- M_s is the middle ordinate, distance from centerline to sight obstruction
- SSD is the stopping sight distance → check table ← design speed
- R_v is the radius of the circular curve, inside lane

Curve Sight Obstruction: Case 1

- Existing Road
  - Known Radius
  - Known M
  - Calculate available SSD
  - Calculate design speed — check table



Curve Sight Obstruction: Case 2

- Existing Road
  - Known Radius
  - Known Design Speed
  - Calculate available SSD
  - Calculate M – stake for trimming

Obstruction Example

- A 2-lane road is to be upgraded to a 4-lane road. The design speed is to increase from 40 to 55 mph. Lanes are to be twelve feet wide and there is no median. The radius of the curve is 800 ft. A bait shop is now located 40 feet inside the centerline of the curve. Does the bait shop need to be moved?

- Radius of center of inner-most lane is:
  Rv = 800 – 12 – 6 = 782
- Sight distance @ 55 mph: 495 feet (Table 7.4)
- Calculate middle ordinate:

$$M_s = 782\left(1 - \cos\frac{90 * 495}{\pi 782}\right) = 38.84 \text{ ft}$$

- Distance of bait shop to middle of the inner-most lane is:
  BSD = 40 – 12 – 6 = 22 ft. (Bait shop is too close.)

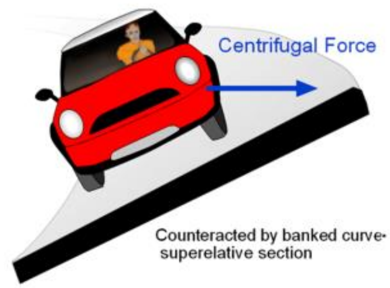
Week9

Common Crown vs. Super-elevation

Cross-sectional view

drainage

Superelevation



$\rightarrow F = \frac{mV^2}{R}$

Superelevation Formula

Decimal

$$e + f_{side} = \frac{V^2}{gR}$$

%

$\rightarrow$ ft/sec $\Rightarrow$ transform to mph

- e is superelevation; f is the assumed coefficient of friction; V is the speed in ft/sec; g is the acceleration of gravity ($32 ft/s^2$); R is the radius of the circular curve in feet.

AASHTO Side Friction (f_s)

American Association of State Highway and Transportation Officials

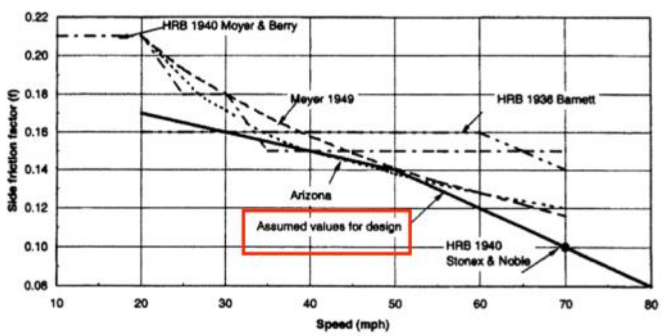


Exhibit 3-13. Side Friction Factors for Rural Highways and High-Speed Urban Streets

Maximum Superelevations

- $e \le 0.10$ on any paved road
- $e \le 0.08$ where there is ice and snow
- $e \le 0.06$ in Wisconsin wherever practical
- $e \le 0.04$ in Wisconsin for urban freeways

2

Design Speed Example

- What is the maximum safe speed for a vehicle traversing a curve with a radius R of 400 feet and a superelevation of 0.08 e. Use AASHTO's design values for the coefficient of side friction (fs=0.15), assuming a target design speed of 40 mph.

$$e + f_{side} = \frac{V^2}{gR}$$

小数

Design Radius Solution

$$0.08 + 0.15 = \frac{V^2}{32.2 * 400}$$

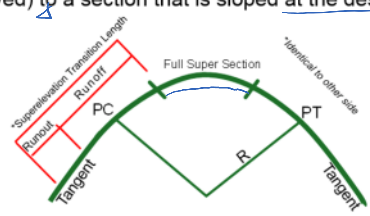
- V = 54.4 ft/sec or 37 mph → excel
- Post a curve speed limit at 35 mph, as it is below the 40 mph speed limit.

54.4 / 1.47

Super-elevation Transition

Tangent runout (TR): Tangent runout is the distance needed to change from a normal crown section to a point where the adverse cross slope of the outside lane or lanes is removed (i.e. the outside lane(s) is level). flat

Superelevation Runoff (L): Supereleration runoff is the distance needed to change the cross slope from the end of the tangent runout (adverse cross slope removed) to a section that is sloped at the design superelevation rate.



Plan View of Superelevation Section

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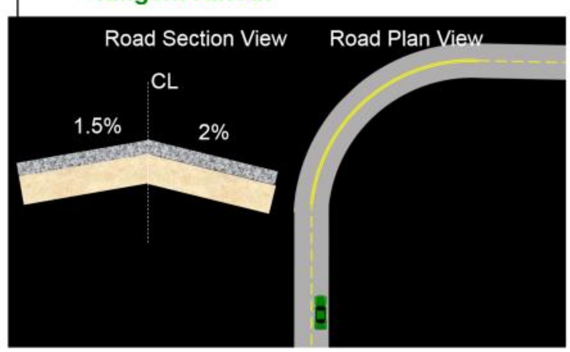
Superelevation Transition -Normal crown

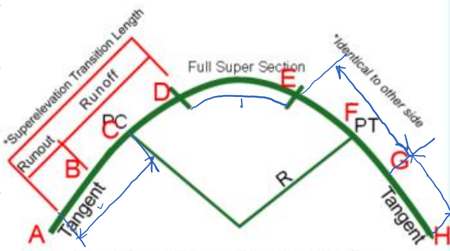
Superelevation Transition -Tangent runout



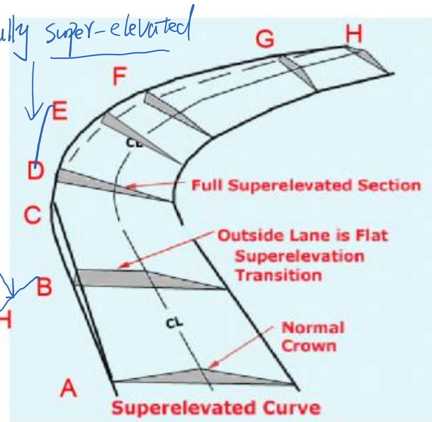
Superelevation Transition -Tangent runout

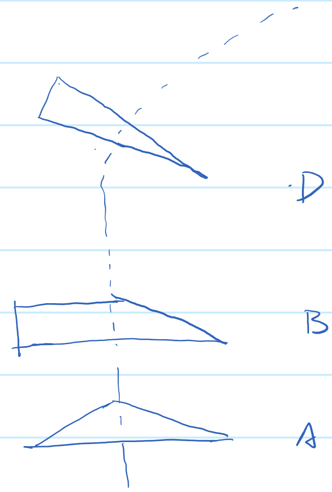
Super-elevation transition

Fully super-elevated



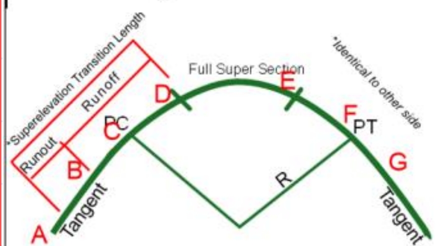
Plan View of Superelevation Section





$L_{C.F} = \frac{\Delta \times R}{5730}$

Super-elevation transition_WisDOT



Plan View of Superelevation Section

Place the tangent runout and two-thirds of the length of runoff on the tangent approach and one-third of the length of runoff on the curve.

$L_{tan} = AB + BC$

$= L_{runout} + \frac{2}{3} L_{runoff}$

$L_{curve} = \frac{1}{3} L_{runoff} + L_{fully} + \frac{1}{3} L_{runoff}.$

$= \frac{\Delta \times R}{5730}$

$L_{tangent}$: AB+BC (BC=2/3*L_{runoff})

L_{curve}: CF=CD+DE+EF (CD=EF=1/3*L_{runoff})

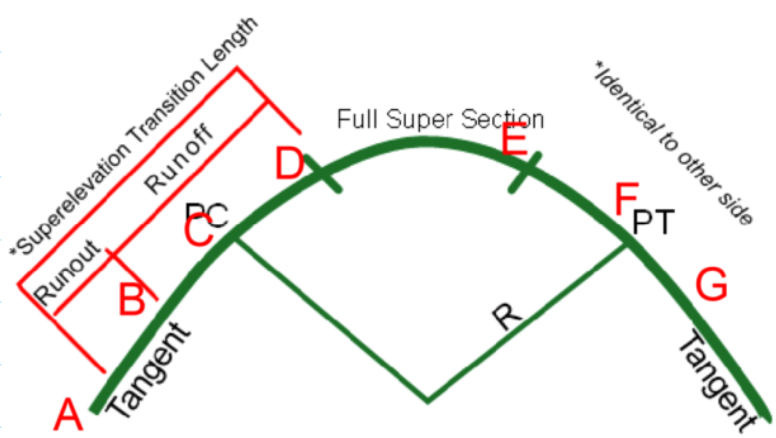
Length of Superelevation Runoff (Length of BD)

- Two-lane Highway
  - L ≅ 30e(V + 32) for 12 foot lanes
  - L ≅ 25e(V + 32) for 10 foot lanes
- Multiply Two-Lane Value by:
  - 1.5 for 4 lanes (2 in each direction)
  - 2.0 for 6 lanes

multiply

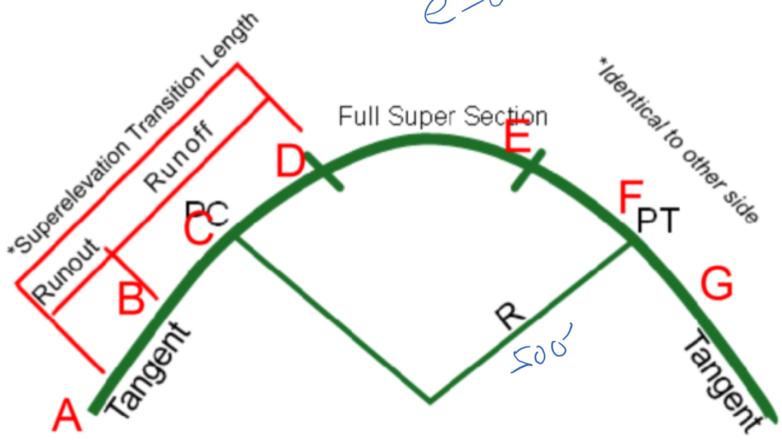
Length of Tangent Runout (Length of AB)

- TR = L * NC/ e
- Where
  - NC is normal crown
  - L is runoff (from B to D)



Plan View of Superelevation Section

$e = 0.08$



Plan View of Superelevation Section

symmetric

Superelevation Runoff Example

- A horizontal curve is being designed on a 4-lane road to have a radius of 500 feet, a superelevation of 0.08, a design speed of 40 mph, at total deflection angle of 60 degrees, and 12-foot lanes. Normal crown is 0.02. (1) What are the superelevation runoff, the (2) tangent runout and the (3) length of curve at full superelevation. Use the Wisconsin standard for the amount of runoff on the tangent.

4 lane 12'

$V = 40$ mph

$\Delta = 60°$

$NC = 0.02$

Superelevation Solution

- $L_{curve} = 60*500/57.3 = 523.60$ ft
- $L_{runoff} = 1.5*30*0.08*(40 + 32) = 259.2$ ft
- $TR_{runout} = 259.2*0.02/0.08 = 64.80$
- Fully superelevated length is $L_{full} = 523.60 - 2* 259.2/3 = 350.7$ ft

(1) runoff $L = 30e(V+32) \times 1.5$

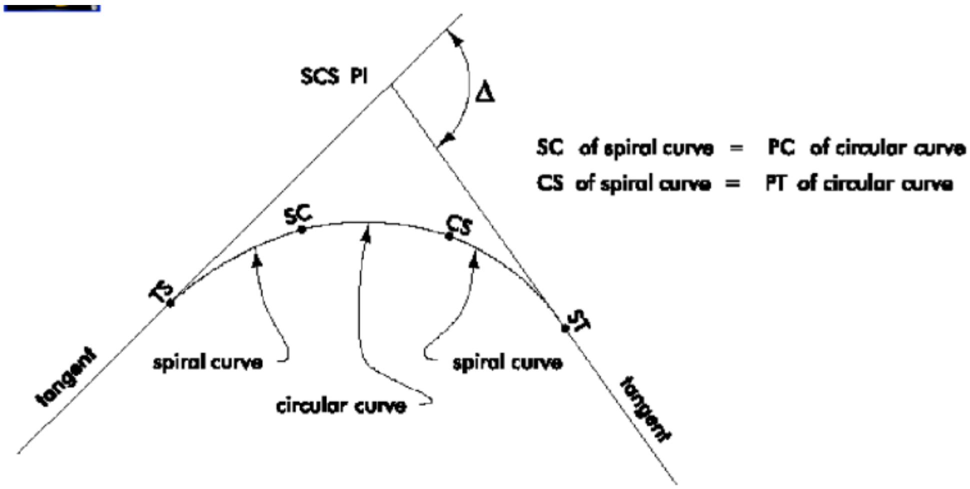
(2) TR_{runout} $TR = L \times NC/e$

(3) length of curve LDE $L_{curve} = CD + DE + EF = \frac{2}{3} \times L + DE = \frac{\Delta \times R}{57.3}$ Degree.

$L_{full\,(DE)} = \frac{\Delta \times R}{57.3} - \frac{2}{3} L_1$

Spirals

a. Provides natural, easy to follow, path for drivers
b. Provides location for superelevation runoff (not part on tangent/curve)
c. Can be used whether the curve is banked or not
d. A smoothly changing radius from a give radius to a tangent section of highway
e. Aesthetic



Minimal Spiral Length

Flat curve (Without superelevation)

$$L_s = \frac{3.15V^3}{RC}$$

Banking curve (With superelevation)

$$L_s = \frac{3.15V}{C}\left(\frac{V^2}{R} - 15e\right)$$

V is in mph; R is the radius of the circular curve, C is the

Flat curve (Without superelevation)

$$L_s = \frac{3.15V^3}{RC}$$

$$L_s = \frac{[illegible]}{C}\left(\frac{[illegible]}{R} - 15e\right)$$

V is in mph; R is the radius of the circular curve, C is the increase in centripetal acceleration (use 2 ft/sec/sec)

5

Spiral Length Example

- Compute the difference of spiral length for a 1200-foot radius curve from using the flat curve formula and the banking curve formula. If the speed is 60mph, C=2 and e=1.2 inches per foot.

Spiral Length Solution

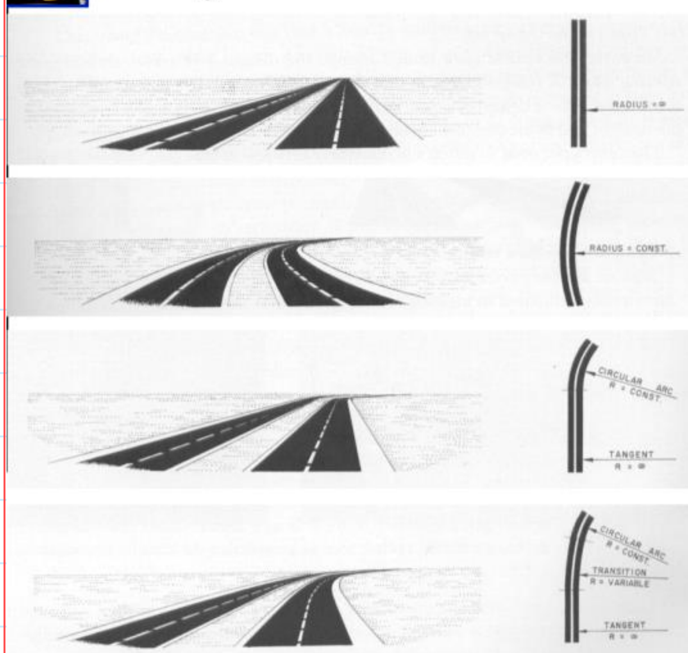
- Flat curve (Without superelevation)

$$L_s = \frac{3.15*60^3}{1200*2} = 283.5\text{ft}$$

- Banking curve (With superelevation)

$$L_s = \frac{3.15*60}{2}\left(\frac{60^2}{1200} - 15*0.1\right) = 141.5\text{ft}$$

Tangents & Curves



Tangent

Curve

Tangent to Circular Curve

Tangent to Spiral Curve to Circular Curve

6---Highway Safety

HCM, highway capacity manual

What does safety really mean?

Highway Safety Has Two Dimensions

Nominal Safety AASHTO Green book	Substantive Safety HSM highway safety manual
Examined in reference to compliance with standards, warrants, guidelines and sanctioned design procedures	The expected or actual crash frequency and severity for a highway or roadway

What do we mean by safety?

Subjective:
- How road users feel?
- How safe we think the design is?

Objective Measure of Safety:
- ① Number or rate (of road accidents/crashes)?
- ② Long term or short term?
- ③ Expected or actual?
- ④ Total or by severity type?

① **Number or Rate?**
- Crash Frequency/Density
  - The total number of crashes per year   Frequency (Road section & Intersection)
  - The number of crashes per mile per year   Density (only Road section)
- Crash Rate

– RMVM: The number of crashes per million vehicle miles per year	(for highway sections)
– RMEV: The number of crashes per million entering vehicles per year	(for highway intersections)

$\frac{\#\ of\ crashes}{ADT \times 365) \times 10^6 \times L}$

$\frac{crashes}{\#\ of\ vehicle \times length\ of\ section}$ ADT AADT

$\frac{crashes}{\#\ of\ vehicles / 2}$

$Rate = \frac{\#\ of\ crashes}{ADT \times 10^6}$

• Example 1_Road section	• Example 2_Intersection
– Location 1: A 6-mile road section has 6 crashes last year. The two-way ADT values was 700 vehicles per day. – Location 2: A 10-mile road section has 9 crashes last year. The two-way ADT values was 1000 vehicles per day.	– Intersection 1 had 13 crashes last year. The four legs of this intersection has two-way ADT values of 9671, 2893, 9506, and 2611 vehicles per day last year. – Intersection 2 that had 16 crashes last year. The four legs of this intersection has two-way ADT values of 9671, 2893, 9506, and 2611 vehicles per day last year.

Example 1:

- Crash Frequency
  - Location 1: 6 vs. Location 2: 9
- Crash Density
  - Location 1: 1 vs. Location 2: 0.9   6/6   9/10
- Crash Rate
  - Location 1: $RMVM = \frac{6crashes}{700vpd * 365days * 6miles} * 10^6 = 3.91$
  - Location 2: $RMVM = \frac{9crashes}{\underbrace{1000vpd}_{ADT \to AADT} * 365days * 10miles} * 10^6 = 2.47$

Example 2:

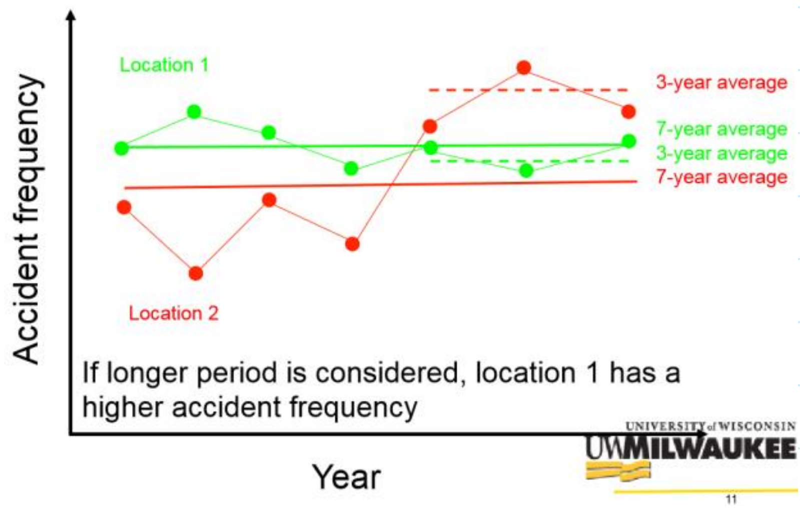
- Crash Frequency
  - Location 1: 13 vs. Location 2: 16

no crash Density

- Crash Rate $RMEV = \frac{Crashes / year}{approachADT * days / year} * 10^6$
  - Location 1: $RMEV = \frac{13}{0.5 * (9671 + 2893 + 9506 + 2611) * 365} * 10^6 = 2.886$
  - Location 2: $RMEV = \frac{16}{0.5 * (12800 + 6803 + 9696 + 4679) * 365} * 10^6 = 2.580$

AHD7

(2) **Long term vs. Short term?**



8-year data.
long term Data.

- Regression to the mean

– the natural variation in crash data
– If regression to the mean is not accounted for, a site might be selected for study when the crashes are at a randomly high fluctuation, or overlooked from study when the site is at a randomly low fluctuation

5-10 years ago
use state wide average crash data

SPF
F = f (ADT, # of lanes, median, control of Intersection)

(3) **Expected or Actual?**

ID	Length	Volume	Total 8-year Accident	Frequency (Crash/Year)	Density (Crash/Mile Year)	Crash Rate (Crash/Hundred Million Vehicle Miles Year)	Frequency Rank	Density Rank	Rate Rank
336	0.122	1472118	323	40.375	330.94	179845.70	1	1	1
39	0.223	981757	60	7.5	33.63	27405.79	2	3	3
461	0.26	931302	55	6.875	26.44	22714.27	3	5	5
482	0.228	1186288	40	5	21.93	14788.87	4	6	7
2	0.336	452421	35	4.3[illegible]	[illegible]	23024.28	5	7	4
491	0.148	1363630	33	4.12[illegible]	[illegible]	16351.43	6	4	6
273	0.507	614498	25	3.125	6.16	8024.38	7	10	10
146	0.23	822534	23	2.875	12.50	12157.55	8	8	8
542	0.44	586176	23	2.875	6.53	8917.58	9	9	9
487	0.032	1339042	18	2.25	70.31	42007.64	10	2	2

cut-off value: 4

Q: How to determine the cut-off value?

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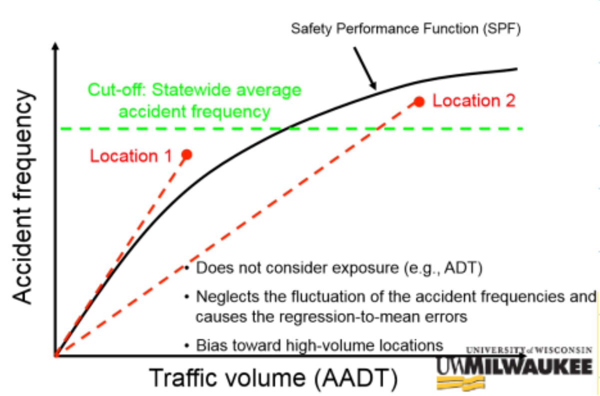
- Safety Performance Functions (SPFs)

– expected average crash frequency as a function of traffic volume and roadway characteristics (e.g., number of lanes, median type, intersection control, number of approach legs).
in a intersection

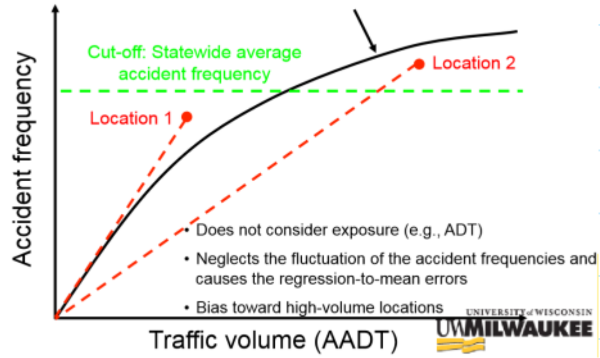
– Their use enables the correction of short-term crash counts.

expected frequency



Q: How to determine the cut-off value?

expected frequency ✓



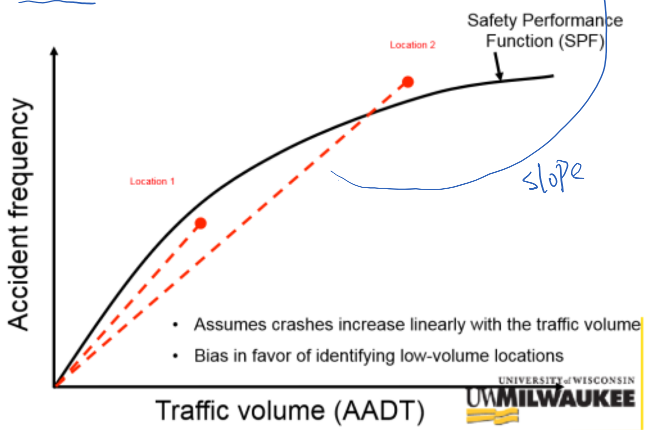
- Does not consider exposure (e.g., ADT)
- Neglects the fluctuation of the accident frequencies and causes the regression-to-mean errors
- Bias toward high-volume locations

8

X

Expected rate

What is the problem by using accident rate as an evaluation index?



Crash Frequency/Density

- Advantages
  – Simple, makes intuitive sense
  – Logical approach if the goal is to reduce the total number of crashes
- Disadvantages
  – Does not consider exposure (e.g., ADT)
  – Neglects the fluctuation of the accident frequencies/densities and causes the regression-to-mean errors
  – Bias toward high-volume locations

4

Total or by severity type?

Site ID	Traffic Volume	Accidents						Rank by total
		Fatal (K)	Incapacitating (A)	Serious Injury (B)	Minor Injury (C)	Property Only (P)	Total	
336	1,472,118	2	2	33	56	230	323	1
39	981,757	0	0	5	13	42	60	2
461	931,302	0	0	4	11	40	55	3
482	1,186,288	0	0	0	6	34	40	4
2	452,421	0	0	4	4	27	35	5
491	1,363,630	0	0	3	2	28	33	6
273	614,498	0	1	3	4	17	25	7
146	822,534	0	1	3	5	14	23	8
542	586,176	1	1	1	0	20	23	9
487	1,339,042	0	0	0	6	12	18	10

Crash Rate

- Advantages
  – Considers exposure to traffic volume
  – Avoids the known bias toward high volume locations in crash frequency/density method
- Disadvantages
  – Assumes crashes increase linearly with the traffic volume
  – Bias in favor of identifying low-volume locations

Crash Severity Methods

- Equivalent Property-Damage-Only (EPDO) Method
  – Weights fatal and injury crashes against a baseline of property damage-only crashes
- Relative Severity Index (RSI) Method
  – Weights the average "comprehensive cost" for crashes at that severity level

- EPDO index

$$EPDO\_Index = W_K K + W_A A + W_B B + W_C C + P$$

W_i = weight for each injury type: K, A, B, and C,
K, A, B, C, P = frequency of crashes under each type: K, A, B, C, and P

$W_K = 200, W_A = 100$ and $W_B = 20, W_C = 2, W_P = 1$

- Relative Severity Index (RSI)

$$RSI = (C_K K + C_A A + C_B B + C_C C + C_P P)/(K + A + B + C + P)$$

C_i = the average comprehensive cost for each injury type: K, A, B, C, and P
K, A, B, C, P = frequency of crashes under each type: K, A, B, C, and P.

Accident Type	Fatal (K)	Incapacitating (A)	Serious Injury (B)	Minor Injury (C)	Property Only (P)
Cost ($)	3000000	208000	42000	22000	2300

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Relative Severity Index (RSI)

Site ID	Traffic Volume	Accidents						RSI	Rank By RSI
		Fatal (K)	Incapacitating (A)	Serious Injury (B)	Minor Injury (C)	Property Only (P)	Total		
336	1,472,118	2	2	33	56	230	323	29606.81	2
39	981,757	0	0	5	13	42	60	7103.03	9
461	931,302	0	0	4	11	40	55	8866.667	8
482	1,186,288	0	0	0	6	34	40	5255	10
2	452,421	0	0	4	4	27	35	9876.667	5
491	1,363,630	0	0	3	2	28	33	9127.273	6
273	614,498	0	1	3	4	17	25	20704.35	3
146	822,534	0	1	3	5	14	23	18444	4
542	586,176	1	1	1	0	20	23	143304.3	1
487	1,339,042	0	0	0	6	12	18	9088.571	7

Bias for low crash frequency locations with disproportionate number of fatal/severe crashes

Accident Type	Fatal (K)	Incapacitating (A)	Serious Injury (B)	Minor Injury (C)	Property Only (P)
Cost ($)	3000000	208000	42000	22000	2300

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Crash Severity Methods

- Advantages
  – Emphasize the severity level of crashes
- Disadvantages
  – Extensive data requirements
  – The arbitrary nature of the weights
  – Bias for low crash frequency locations with disproportionate number of fatal/severe crashes

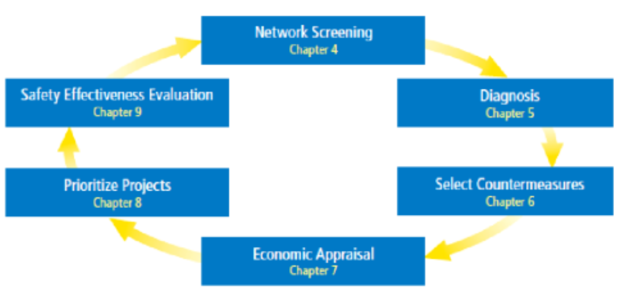
Substantive Safety

- Hazardous Locations are Not Necessarily High-Crash Locations
- Best measured in terms of expected long term average crash frequency by severity types

SPF 8 years not rate

Roadway Safety Management Process

(From Highway Safety Manual)

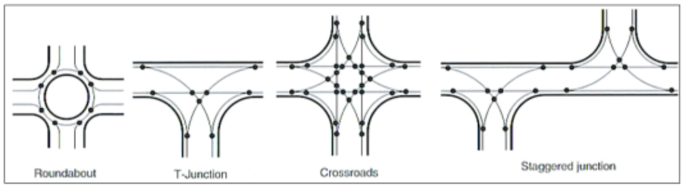


Week10--Traffic Operation -- Traffic Signals

Why Traffic Signals?

Purpose

1. Conflicting traffic movements, make roadway intersections unsafe for vehicles and pedestrians



2. Intersections are a major source of crashes and vehicle delay (as vehicles yield to avoid conflicts with other vehicles)

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History of Signalized Intersections

1868 - London, UK
- Manually operated semaphores (flags)

1914 - Cleveland, OH

- 1st Electric Signal

1917 - Detroit, MI
- Amber color introduced

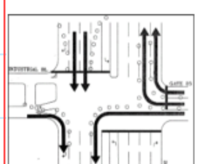
buffer time

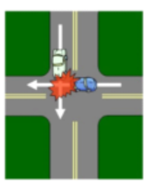
extra time to pass

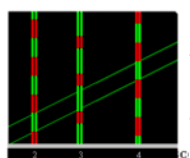
UWM

Traffic Signals : Advantages

- Orderly flow of traffic at intersection by avoiding conflicts;
- Reduce the frequency and severity of certain types of crashes, especially right-angle collisions;
- Be coordinated to provide for continuous movement of traffic at a definite speed along a given route under favorable conditions.







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Green Wave

Traffic Signals : Disadvantages

- Large stop time delay
- Complex signal design problems

The possible effects of a poorly-timed traffic signal :
- increase in vehicle delay
- increase in vehicle crashes (particularly rear-end crashes)

What do they Cost?

- Annual Maintenance: \$3,000 - \$8,000
  - Labor and Bulb replacement
- Electricity: ≈\$1,400/yr
- Update Signal timing: \$2,500 - \$3,100 per intersection per update
  - Should be done often – cheap way of improving operations, but Labor-intensive
- Upgrade signal: \$10,000 per intersection
  - Should be done every 10 years (National Traffic Signal Report Card)

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1. material
2. labor
(Installment & maintainance)

When is a Traffic Signal Warranted?

- Manual on Uniform Traffic Control Devices (MUTCD)
  - Available free online: http://mutcd.fhwa.dot.gov

- National Minimum Standard for all traffic control devices on streets, highways and bicycle trails.

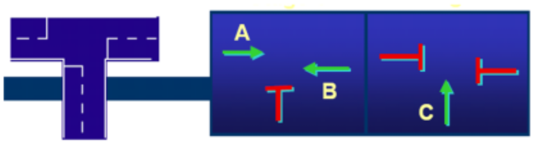
2

1,

Basics

- I. Phase and cycle length

Phase A Phase B



Phase: allotted time to a movement

I. Phase and cycle length

- Sequence of phases
  - Green phase (G)
  - Amber (yellow) phase (Y)
  - Red phase (R)
- Cycle length (C) – time from start of green phase to the next green phase (typically 45-180 s)

$$C = G + Y + R$$

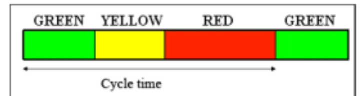


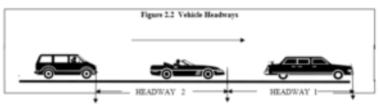
Figure 1: signal timing at a traffic signal

Basics

2,

- II Discharge headway, saturation flow

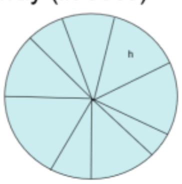
Headway & Flow Rate



⇒ freeway segment

Headway (in secs) ⟶ Flow rate

\# of vehicles in 1 hour



1 hour

$$q = 3600 / \bar{h}$$

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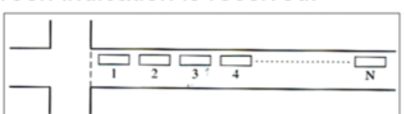
Discharge Headways

- Consider N vehicles discharging from the intersection when a green indication is received.



Vehicles in an intersection queue.

- *The first discharge headway* is the time between the initiation of the green indication and the rear wheels of the first vehicle to cross over the stop line.
- *The Nth discharge headway* (N>1) is the time between the rear wheels of the N-1th and Nth vehicles crossing over the stop line.

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$h_1 = 4s$ reaction time + drive time

$h_2 = 3s$ overlap

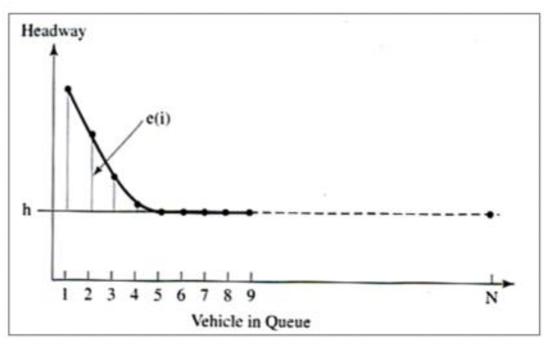
$h_3 = 2.5s$

$h_4 = 2.3s$

⋮

$h_N = 2.3s$

- The *headway* begins to level off with 4 or 5th vehicle.
- The level headway *h = saturation headway*



saturation flow

$$S = \frac{3600}{h_s}$$

3

Saturation flow rate

In a given lane, if every vehicle consumes an average of h seconds of green time, and if the signal continues to be uninterruptedly green, then S (unit: vph) could enter the intersection, where S is the saturation flow rate (vehicles per hour of green time per lane), given by

$$S = \frac{3600}{h}$$

Saturation flow rate

- The Highway Capacity Manual (HCM) 2000 suggests 1900 vphpl as the ideal saturation flow rate
- This baseline can be increased or decreased depending upon the situation Factors such as:
  - Lane width
  - Grade
  - Pedestrians
  - On-street parking

3.

Basics

- III Loss Time

Loss Time: A portion of green or yellow time which is not utilized by traffic flow movements in a cycle.

Start-up loss time: At the beginning of each green indication as the first few cars in a standing queue experience start-up delays. This delay is measured as start-up loss time.

The clearance loss time: It is estimated by the amount of the yellow time not used by vehicles.

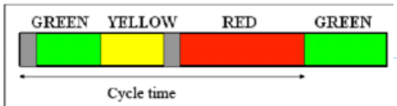


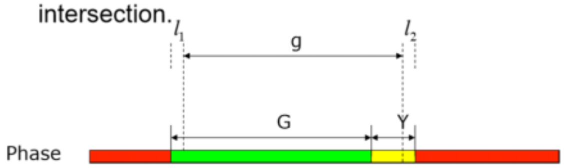
Figure 1: signal timing at a traffic signal

4.

Basics

- IV Effective green

The effective green time, for a phase, is the time during which vehicles are actually discharging through the intersection.



G: Displayed green time for the phase

Y: Yellow time for the phase

t_L : Total loss time for the phase

Effective Green Time: $g = G + Y - t_L$ $\quad t_L = l_1 + l_2$

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5.

Basics

- V Movement capacity

The max number of vehicles can be processed at the intersection through the movement i

$$\text{Movement i} \longrightarrow c_i = S_i \frac{g_i}{C}$$

gi: effective green time for movement i

Si: saturation flow rate for movement i

C: cycle length

4

Basics

- VI Phase plan – Permitted vs. Protected

- **Permitted left-turn movement:**
  A movement that is made through a conflicting vehicle movement.

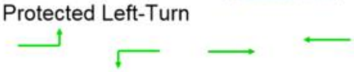
- **Protected left-turn movement:**
  A movement that is made without conflict with other movements. The movement is protected by traffic control signal design with a designated green time for the specific movement.

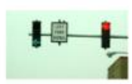
exclusive area .

Basics

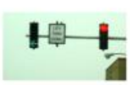
- VI Phase plan – left turn treatment
- Permitted Left-Turns

- Protected Left-Turn





- Protected/Permitted or Permitted/Protected



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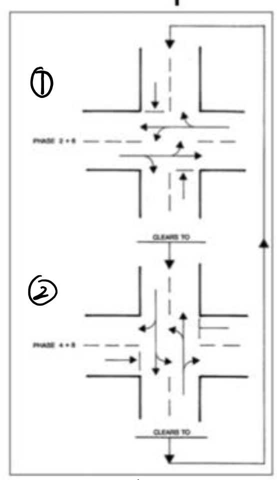
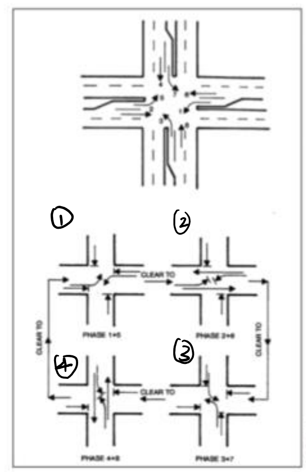
- VI Phase plan



two phase



Four phase

Goals of Signal Timing

- Minimize the # of Green phases ⇒ ↓ lose time.
- Maximize the # of vehicles moving through the intersection during all green phases
- Minimize delay (shorter cycle lengths)
- Maximize capacity (longer cycle lengths)

Week 11--Signal timing

Types of signals

Actuated v.s Adaptive

Pretimed 1st generation

accutated 2nd generation.

semi Fully

Adaptive 3rd generation — system optimal / overall decision

ex. WI 100

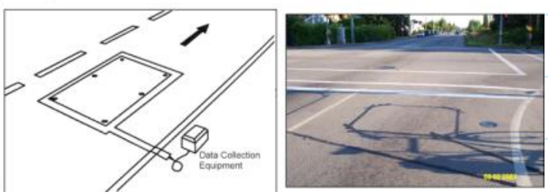
Pretimed

- Lengths of Phases predetermined and statically set
- Strengths
  - Simplest
  - Less infrastructure (read: maintenance)
- Weakness
  - Can not account for cycle-to-cycle variation of traffic
  - Will remain fixed until updated (which costs $)

UWMILWAU

Actuated Signals

- Most use Inductive-Loop Detectors in pavement



Pre-timed vs. Actuated

- Pre-timed
  - Lower maintenance resources
  - Consistent demand
  - Simpler
- Actuated
  - Significant variation in demand
  - High volume meets low volume road
  - Greater control

Signal Timing Design

- Procedures
- Collect traffic volume and composition
- Development of a phase plan and sequence (Left-turn treatment)
- Determine the lane groups
- Convert all volumes in a lane group into "equivalent through" traffic per lane
- Determine the total critical **lane** volume to be served
- Determination of cycle length
- Splitting the green time
- Safety requirements

Procedure

% or hv or bus ⇒ affect discharge rate / headway

① collect traffic volume & composition

② Develop a phase plan

①

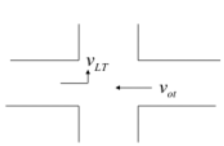
Signal Timing Design

- Left-turn treatment (HCM 2000)

$v_{LT} \geq 200 veh/h$ OR

$v_{LT} \cdot (v_{ot}/N_{ot}) \geq 50{,}000$

Need a protected left-turn phase



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① Left turn treatment

Critria:

$V_{LT} > 200$ veh/h

$V_{LT} \cdot (V_{ot}/N_{ot}) \geq 50{,}000$

left-turn ONLY signals when "green arrow"?

② Phase plane

2

Friday, December 9, 2016 1:47 PM

2

- Determine lane groups

a b c d

A lane group is a lane, or adjacent set of lanes that accommodates one or more traffic movements in a homogeneous manner

→ Highway Capacity Manual

3

- Traffic volume conversion
- Why we need the conversion?
- Protected LT : 1 LT = 1.05 TCU
- Permitted LT : depending on the opposing volume
- RT = 1.32 TCU, depending on pedestrian volume in conflicting crosswalk
- Equivalent through volume for a lane group

$$V_{EQL} = \frac{V_{LTE} + V_{TH} + V_{RTE}}{N}$$

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④ Traffic volume conversion

Protected LT : 1 LT = 1.05 TCU   Through car unit

Permitted LT : — Table.

RT = 1.32 TCU

$$V_{EQL} = \frac{V_{LTE} + V_{TH} + V_{RTE}}{N}$$ Lane volume

4

- Cycle length – basic relation

$t_L = l_1 + l_2$ ← Lost time per phase

$L = N \cdot t_L$ ← Lost time per cycle

Num of phases

$L_H = L \cdot (3600/C)$ ← Lost time per hour

$T_g = 3600 - L_H$ ← Total effective green in an hour

$V_c = T_g / h = \frac{[3600 - N \cdot t_L \cdot (3600/C)]}{h}$ Max lane volume can be served

- Minimum cycle length

$C_{min} = \frac{N \cdot t_L}{1 - \frac{V_c}{3600/h}}$ ← If the lane volume to be served is known

- Designed cycle length

$C_{des} = \frac{N \cdot t_L}{1 - \frac{V_c / PHF}{(3600/h) \cdot (v/c)}}$ ← Not the optimal cycle length

⑤ cycle length — basic relation
loss time based.

per phase
{ start-up loss l_1
{ part of yellow time l_2 > t_L

per cycle $L = N \cdot t_L$

$3600/C$ # of cycles/hour

$V_c = \frac{T_g}{h}$

$T_g = 3600 - N \cdot t_L (3600/C)$

$C_{min} = \frac{N \cdot t_L}{1 - \frac{V_c}{3600/h}}$ — Minimum cycle length.

$C_{des} = \frac{N \cdot t_L}{1 - \frac{V_c/PHF}{(3600/h) \cdot (V/C)}}$ — Designed cycle length.

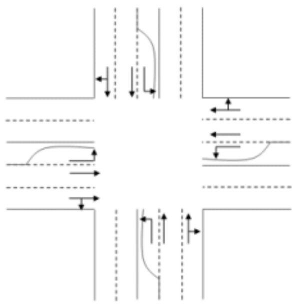
→ % of capacity want to use

5 ***Allocation of green time***

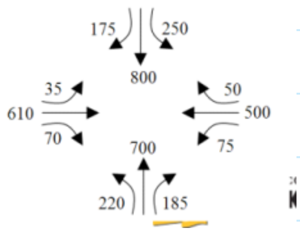
Based on the distribution of critical lane volume on each phase

→ under-saturated condition

- # Example



PHF =0.92
(v/c) = 0.90
Moderate numbers of pedestrians



- # Solution

Step-1: If protected phase for LT is needed?

(a) $V_{LT} < 200$ vph OR

(b) $V_{LT} \times \dfrac{\text{Opposing TH}}{\text{No. of lanes}} < 50{,}000$ vph

AND

(Guidelines for LT Phase)

Eastbound Approach $V_{LT} = 35 < 200$ vph
x-prod $= 35 \times 500/2$
$= 8{,}750 < 50{,}000$ NO

Westbound Approach $V_{LT} = 75 < 200$ vph
x-prod $= 75 \times 610/2$
$= 22{,}875 < 50{,}000$ NO

Southbound Approach $V_{LT} = 250 > 200$ vph YES

Northbound Approach $V_{LT} = 220 > 200$ vph YES

Step-2 : Convert all volumes into "through"- passenger car units

- Protected LT : 1 LT = 1.05 TCU
- Permitted LT : depending on the opposing volume
- RT = 1.32 TCU

Table 18.1

TCU Volumes for Sample Problem 1

Approach	Movement	Volume (vph)	Equivalent	Volume (tcu)	Shared Lane Grp (tcu)	No. Lanes
EB	LEFT	35	4.00	140	140	1
	THROUGH	610	1.00	610	702	2
	RIGHT	70	1.32	92		
WB	LEFT	75	5.00	375	375	1
	THROUGH	500	1.00	500	566	2
	RIGHT	50	1.32	66		
SB	LEFT	250	1.05	263	263	1
	THROUGH	800	1.00	800	1031	2
	RIGHT	175	1.32	231		
NB	LEFT	220	1.05	231	231	1
	THROUGH	700	1.00	700	944	2
	RIGHT	185	1.32	244		

Step-3 : Design the signal phasing plan

Φ_A	231	263	V_{cA}=263 tcu's/ln
Φ_B	472	517	V_{cB}: 517 tcu's/ln
Φ_C	140 / 351	283 / 375	V_{cC}=375 tcu's/ln

Summation of the highest volume in each phase

$V_c = 263$
517
375
1,155 tcu's

Critical volume 1,155 tcu ⇒

3phases, Loss time 3 seconds

Step-4 : Computation of the min. cycle length

$$C_{min} = \frac{3\times 3}{1-\dfrac{1155}{1615(0.92)(0.90)}} = 66.2\text{sec} \approx 70\text{sec}$$

Step-5 : Allocation of green time, based on the distribution of critical lane volume on each phase

Total effective green time= 70sec - 3×(3sec)= 61sec

Loss time

Then,

$$G_A = (\frac{263}{1155}) \times 61 = 13.9\text{sec} \approx 14$$

$$G_B = (\frac{516}{1155}) \times 61 = 27.3\text{sec} \approx 27$$

$$G_C = (\frac{375}{1155}) \times 61 = 19.8\text{sec} \approx 20$$

61

4---public transit

What is "transit"?
Collective Transportation

GOAL: TO MAXIMIZE SPACE
To maximize the usage of space

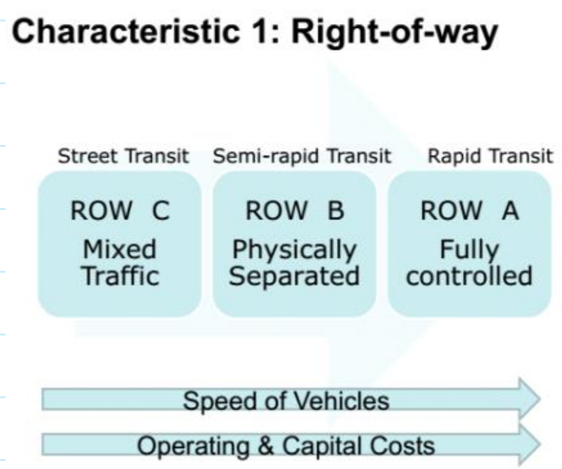
in average
vehicle capacity: Car (occupying 1.2 people per car.
Bus 40 people   1 Bus = 30 cars
Road space   1 Bus = 2.5 cars
Fuel consumption 1 Bus = 3 cars

1. What is transit? collect people / space / enviroment

① collect people ② for general public

2. Transit Mode Definition

4 Basic characteristic



4 Basic Characteristics:
- Right-of-Way (ROW)
- System Technology
- Types of Service
- Organizational Oversight

2) system technology ①
②
③
④

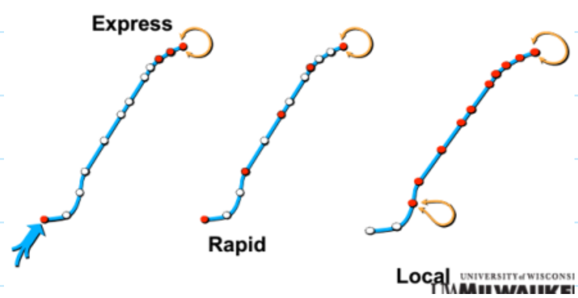
Characteristic 2: System Technology

Support: Vertical contact	Guidance: Lateral control	Propulsion: Power system	Control: Spacing
• Rubber tire on pavement • Steel wheel on rail • Vehicle on water, mag lev, etc.	• Steered by driver • Guided by track	• Diesel engine • Electric motor • Hybrid • Magnetic forces, etc	• Manual/visual • Manual/ signals • Automatic

5

Characteristic 3: Types of Service

3) types of surface { ① Express ② Rapid ③ Local



Characteristic 4: Organization Oversight

4) orgnization oversight { individual's on-demand.
public Group / fixed schedule route

On-Demand Individuals

Public Group

4 Major Transit Families

- Street Transit
  - Right-of-Way (ROW)
  - System Technology
  - Types of Service
  - Organizational Oversight

- Rapid Transit
  - Right-of-Way (ROW)
  - System Technology
  - Types of Service
  - Organizational Oversight

· A steel/riber + Express / ripid

- Semi-rapid Transit
  - Right-of-Way (ROW)
  - System Technology
  - Types of Service
  - Organizational Oversight

· B steel / riber

- Paratransit
  - Right-of-Way (ROW)
  - System Technology
  - Types of Service
  - Organizational Oversight

Ferry/Zipper/ & specialized.

4. 4 Major Transit Family

1) street transit modes.

① • Regular Bus
② • Express Bus
③ • Trolleybus
④ • Streetcars(trolleys)

2) Semi Rapid Transit mode.

- Bus Rapid Transit
- Light Rail Transit
- Automated Guideway Transit

Bus Rapid Transit BRT
Light Rail Transit
Automated Guideway Transit B

3) Rapid Transit Modes

- Rail Rapid Transit (Metro)
- Light Rail Rapid Transit
- Rubber-tired Rapid Transit
- Monorail
- Regional (Commuter) Rail

Light rail rapid system

4) Paratransit Modes

- Taxicabs
- Car Sharing
- Dial-a-ride
- Jitney / Publico

Para — alongside the road
supplemental service for those convention transit cannot cover.

2011 APTA Modal Facts

APTA

American Public Transportation Association (APTA)

Table 5: Unlinked Passenger Trips and Passenger Miles by Mode, Millions
Report Year 2011

Mode of Service	Passenger Trips		Passenger Miles	
	Millions	Percent	Millions	Percent
Bus	5,191	50.3%	20,408	36.4%
Bus Rapid Transit	6	0.1%	23	< 0.1%
Commuter Bus	37	0.4%	984	1.8%
Commuter Rail	466	4.5%	11,427	20.4%
Demand Response	191	1.9%	1,580	2.8%
Ferryboat	80	0.8%	416	0.7%
Heavy Rail	3,647	35.3%	17,317	30.9%
Hybrid Rail	6	0.1%	70	0.1%
Light Rail	436	4.2%	2,203	3.9%
Other Rail Modes (a)	44	0.4%	47	0.1%
Publico	39	0.4%	172	0.3%
Streetcar	43	0.4%	96	0.2%
Transit Vanpool	34	0.3%	1,176	2.1%
Trolleybus	98	0.9%	160	0.3%
Total All Modes	10,319	100.0%	56,077	100.0%

(a) Aerial tramway, automated guideway transit, cable car, inclined plane, and monorail.
Unlinked Passenger Trips by Mode data from 1902 through 2011 can be found in the *2013 Public Transportation Fact Book,*

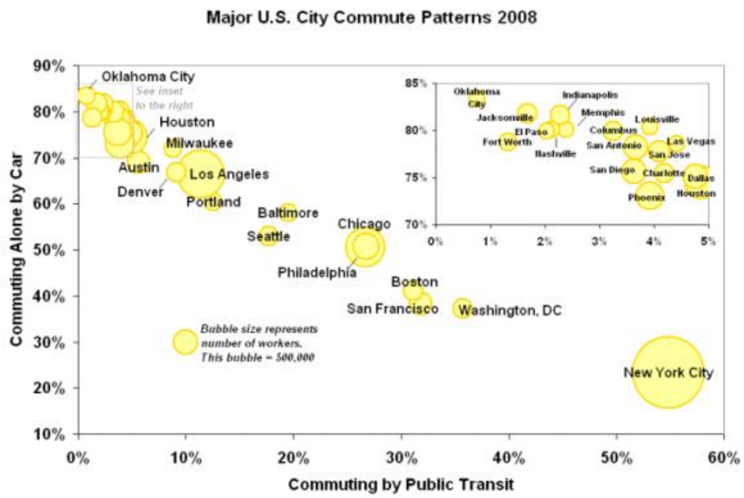
Source: APTA, 2013, Public Transportation Fact Book

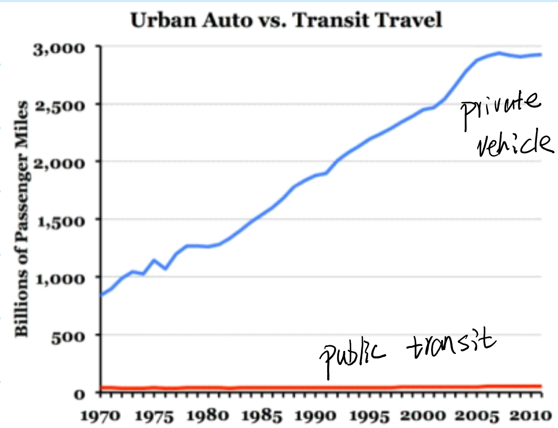
Unlinked Passenger Trips *is the number of times passengers board public transportation vehicles. Passengers are counted each time they board vehicles no matter how many vehicles they use to travel from their origin to their destination and regardless of whether they pay a fare, use a pass or transfer, ride for free, or pay in some other way.*

Week12---**Elements of good transit service**

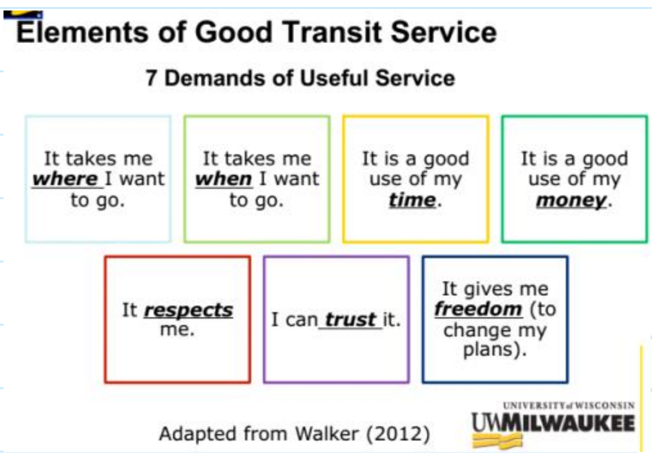
Is Public Transit competitive today?

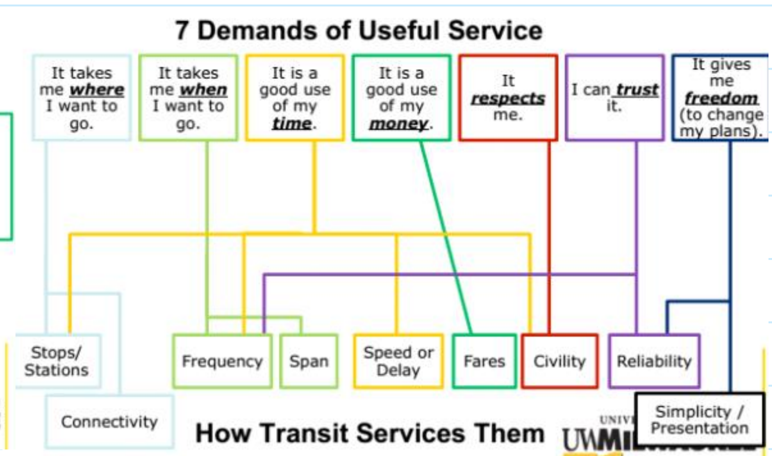
① Longer travel time
② Not easy to access
③ Inconvenient to transfer





WHAT ARE THE KEY ELMENTS OF GOOD TRANSIT SERVICE?





①

stations

accessibility

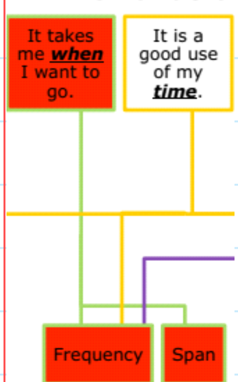
2 ## Impact of Street Network

It takes me ***where*** I want to go.

IMAGES AND MAPPING COURTESY OF URBAN DESIGN 4 HEALTH

connectivity

2



Frequency

It takes me ***when*** I want to go.

- ***Frequency*** is the number of transit vehicles in a given period of time. (in vph)
- ***Headway*** is the measurement of time between transit vehicles. (in min.)

  Headway = 60 / Frequency

- Frequency generally divided into 2 categories:
  - *High Frequency Service:* Headways less than 10 minutes
  - *Low Frequency Service:* Headways greater than 10 minutes

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Frequency & Wait Time

It takes me ***when*** I want to go.

- Frequency is important because it determines waiting times.
- Wait time for high stable frequency transit is, on average, half of the headway.

  Wait Time = Headway/2

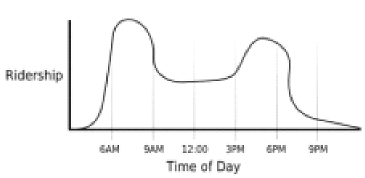
  Wait time vs. travel time in transit?

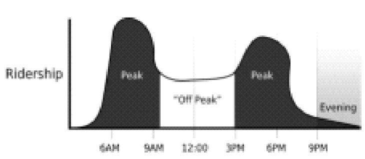
  1 min waiting ≈ 2 min riding

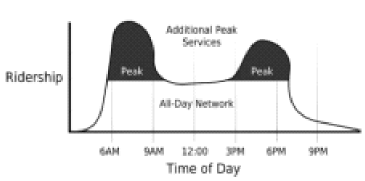
Span

Span is the length of time that Transit service runs.

Spans can be all day or can have peaks.

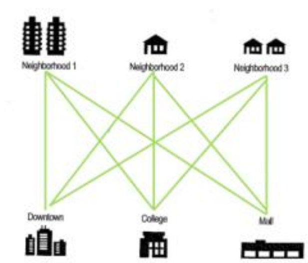






Simple City

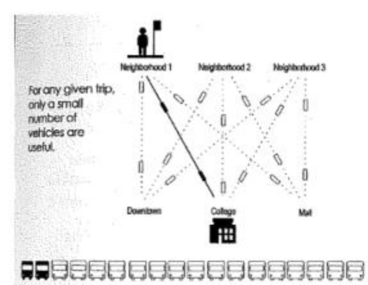
Direct service



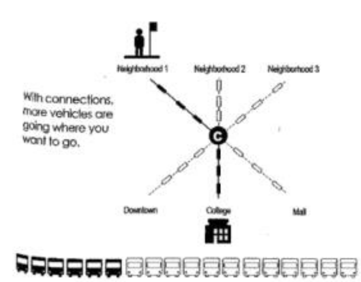
Transfer station

Frequency

Direct Service Option



Connective Option



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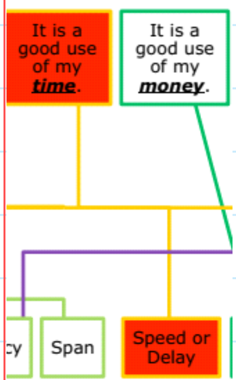
(3)



Speed and Delay

- The speed of transit service determines its travel time
- ☆ Transit Priority
  - Prioritize personal mobility versus vehicle mobility
  - Ensure speed and reliability for transit

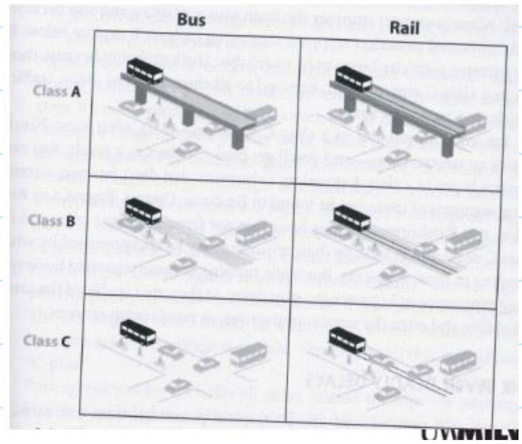
Right-of-way

Street Transit	Semi-rapid Transit	Rapid Transit
ROW C Mixed Traffic	ROW B Physically Separated	ROW A Fully controlled

semi-control

speed
cost

UWMILW



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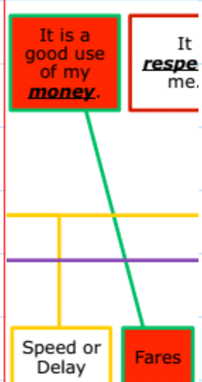
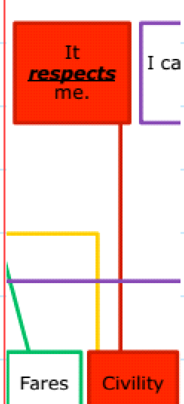


Fares

- Typical farebox recovery ~ 35%
- Other funds are local, state and federal subsidies
- ALL transportation is subsidized
- Some of these subsidies are hidden
  - "Free" parking
  - Public health impacts
  - Pollution
  - Oil dependence

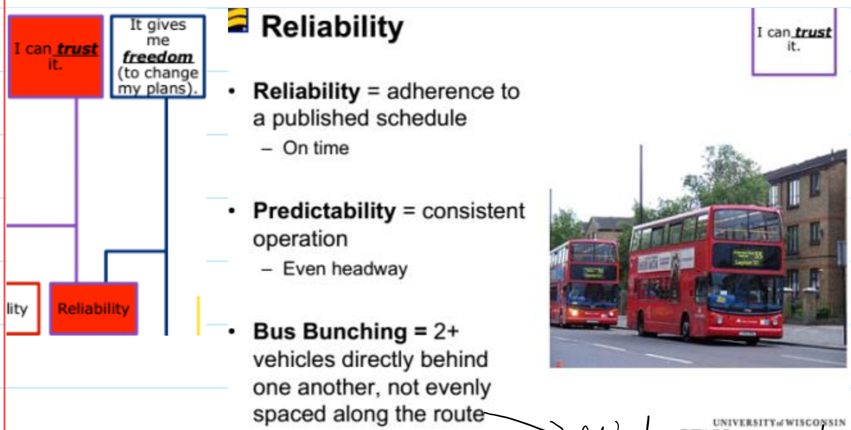
(5)



Civility

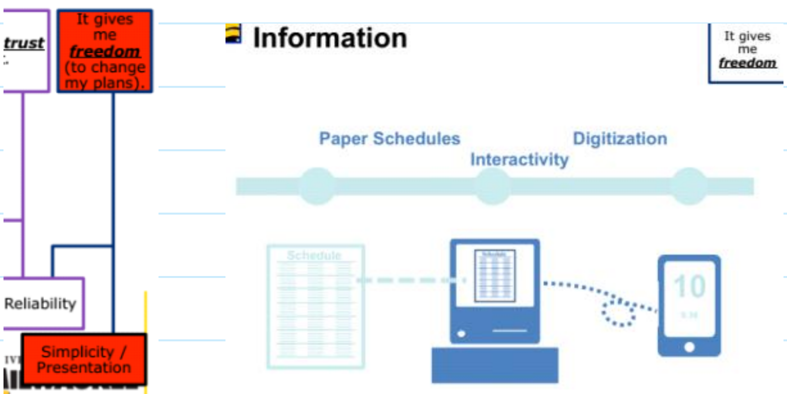
- How do you perceive transit?
- Why should transit be perceived as *cool*?
- How do we make transit *cool*?

6



aviod in schedualing

7



Summary

- In an urban environment, we simply do not have enough space to travel individually in cars.
- Transit must be given the highest possible ROW category to ensure personal mobility rather than vehicular mobility.
- Proper planning, design and operations are necessary.
- The street network has to be properly designed.
- Information must be presented to take advantage of new technology.

Mid term 2 / hw 9

planning — 4 steps

2 calculations
2 concepts

Design
- vertical
  - parameters
  - SSD
- horizontal
  - parameters
  - super elevation

Softy — 1 short question

Trip distribution — conceptual
Mode split — caculation
Traffic assignment

Highway Design.
- Functional class
- Geometric Design

Vertical Aligment
how to get
a, b, c
Parabola curve

Ms — cal

Superelevation Formula

1. Definition
2. concise conceptual
3. Calculation (2) Softy & Signal.
4. Graphic (functional road map
   Transit network design)

4

2016年9月25日 23:54

6

4 Caculators. ① parameters flow

② Greensheild mold.

③ Queuing Theory — Work Zone

④ Los

REVIEW

- TOPIC 2_TRAFFIC FLOW
  – Parameters
    - Macro: Volume, Speed & Density
    - Micro: headway, speed & spacing
  – Models
    - Relations of Volume, Speed & Density
      – Fundamental equation
      – Greenshield's & Non-linear
    - Possion distribution
      – Left-turn bay length
      – Headway

1 calculation

REVIEW

- TOPIC 3_HIGHWAY CAPACITY
  – LOS (LEVEL OF SERVICE)
    - SUPPLY
    - DEMAND
    - LOS DETERMINATION
  – QUEUE SYSTEM
    - QUEUEING DIAGRAM
    - QUEUE MODELS

What is LOS

procedure.

conceptual

- TOPIC 4_TRANSPORTATION PLANNING
  – 4-STEP TRAVEL FORECAST
    - TRIP GENERATION
    - TRIP DISTRIBUTION
    - MODAL SPLIT
    - TRAFFIC ASSIGNMENT

conceptual

- TOPIC 5_HIGHWAY DESIGN
  – VERTICAL CURVE DESIGN
    - Vertical curve alignment, elevation, stationing, vertical curve length requirement
  – HORIZONTAL CURVE DESIGN
    - horizontal curve alignment, stationing, design speed at the horizontal curve, middle ordinate at the horizontal curve, superelevation calculation, and superelevation run-off and tangent run-out determination

Defination

conceptual

- TOPIC 6_HIGHWAY SAFETY
  - What do we mean by the safety?
    - Number or rate (of road accidents/crashes)?
    - Long term or short term?
    - Expected or actual?
    - Total or by severity type?

- TOPIC 7_TRAFFIC OPERATIONS
  – Signalized intersections: advantages and disadvantages of signals, cycle, phase, discharge headway, saturation flow, start-up loss, effective green, movement capacity, left-turn treatment, ring-and-barrier structure, split phase, signal timing design for isolated intersections

1 calculation
signal timing
+
conceptual

- TOPIC 8_PUBLIC TRANSIT
  – What is transit?
    - Definitions, characteristics, types
  – Why do we need transit?
  – What are the key elements of good transit service?
    - 7 Demands of Useful Service
    - How transit service these demands?

Drawing.
public network

Final exam

- Definitions (10 items, 20%)
- Short answers (5/7 items, 35%)
- Calculations (2 items, 30%)
- Graphical illustrations (1 item, 15%)

7

2016年11月7日 8:16

Homework

2016年9月14日 11:41

Question 1).

$$D = \frac{5280}{\bar{d}} \quad (1)$$

when $O = 100\%$, Distance headway $\bar{d} = L_v + L_d$. (2)

(1)(2) $\Rightarrow$ $D = \frac{5280}{L_v + L_d}$

when $O \neq 100\%$, $\bar{d} = \frac{L_v + L_d}{O}$ (3)

(1)(3) $\Rightarrow$ $D = \frac{5280 \times O}{L_v + L_d}$

Question 2).

	old	new
when $S = 10$ MPH,	$d_1 = 18'$	$d_3 = 14.67\ ft/s \times 2 = 29.34'$
when $S = 50$ MPH,	$d_2 = 18' \times 50/10 = 90'$	$d_4 = 5 \times 14.67\ ft/s \times 2 = 146.7'$

$\because$ $d_3 > d_1$ and $d_4 > d_2$

$\therefore$ the newer time rule of thumb is more conservative.

Question 3).

① identify the peak hour, which is 4:45–4:50 to 5:40–5:45

② divide it to 4 part, sum the flow rate for each 15 min,

③ chose the max flow rate, use $PHF = \frac{V}{4 \times 15}$ to calculate

Time period	Volume/vehicles
4:30–5;30	18700
4:35–5:35	18800
4:40–5:40	18750
4:45–5:45	18810
4:50–5:50	18380
4:55–5:55	18440
5:00–6:00	18350

Table 1

模板

2016年9月25日 23:54

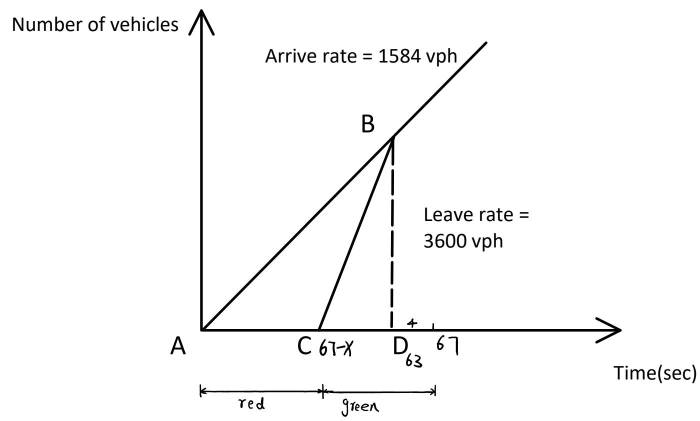


Table2.

Time	Flow rate	Flow rate/15 min	Flow rate/ 1 hour	PHF
4:45-4:50	1780			
4:50-4:55	1720			
4:55-5:00	1540	5040		0.93
5:00-5:05	1670			
5:05-5:10	1650			
5:10-5:15	1700	5020		
5:15-5:20	1400			
5:20-5:25	1500			
5:25-5:30	1700	4600		
5:30-5:35	1300			
5:35-5:40	1300			
5:40-5:45	1550	4150	18810	

∴ PHF is 0.93 for the peak hour 4:30 – 5:30

Question 4)

a) $D = \frac{40+1}{0.78 \times 3} = 17.52$ veh/mile/lane

b) Flow rate $q = D \times S = 17.52 \times 52 = 911$ veh/hr

Q1

2016年10月22日 22:52

Demographic forecast				
Person per Household	Vehicles per Household			
	0	1	2	3+
1	100	300	150	0
2	110	250	50	30
3	90	250	50	40
4	150	210	60	50
5+	20	50	50	20

Trip rate				
Person per Household	Vehicles per Household			
	zero	one	two	three +
1	2.6	4	4	4
2	4.8	6.7	8.1	8.4
3	7.4	9.2	10.6	11.9
4	9.2	11.5	13.3	15.1
5+	11.2	13.7	16.7	18

Trip production =	16344			
Person per Household	Vehicles per Household			
	zero	one	two	three +
1	260	1200	600	0
2	528	1675	405	252
3	666	2300	530	476
4	1380	2415	798	755
5+	224	685	835	360

Q2

2016年10月22日　　22:52

Step 1: Data Input

t_{ij}	Site 1	Site 2	Site 3	
City A	6	4	2	travel time
City B	2	8	3	
City C	1	3	5	

City	Average # of Skiers
City A	250
City B	450
City C	300
sum	1000

P

Ski Site	Attraction (k=1)	Adjusted (k=2)
Site 1	395	353
Site 2	180	277
Site 3	425	409
sum	1000	

A

Step 2: friction matrix Fij

F_{ij}	Site 1	Site 2	Site 3
City A	26.00	41.00	52.00
City B	52.00	13.00	50.00
City C	82.00	50.00	39.00

Step 2: friction matrix Fij

F_{ij}	Site 1	Site 2	Site 3
City A	26.00	41.00	52.00
City B	52.00	13.00	50.00
City C	82.00	50.00	39.00

Step 3: Aj*Fij

A_jF_{ij} (k=1)	Site 1	Site 2	Site 3	Sum
City A	10270.00	7380.00	22100.00	39750.00
City B	20540.00	2340.00	21250.00	44130.00
City C	32390.00	9000.00	16575.00	57965.00

A_jF_{ij} (k=2)	Site 1	Site 2	Site 3	Sum
City A	9184.67	11367.81	21275.63	41828.11
City B	18369.34	3604.43	20457.34	42431.10
City C	28967.04	13863.19	15956.72	58786.94

Step 4: Tij

T_{ij} (k=1)	Site 1	Site 2	Site 3	Production
City A	65	46	139	250
City B	209	24	217	450
City C	168	47	86	300
Computed Attraction	442	117	441	
Given Attraction	395	180	425	
convergence (<=5%)?	12%	35%	4%	NO

T_{ij} (k=2)	Site 1	Site 2	Site 3	Production
City A	55	68	127	250
City B	195	38	217	450
City C	148	71	81	300
Computed Attraction	398	177	426	
Given Attraction	395	180	425	
convergence (<=5%)?	1%	2%	0%	YES

Choose Doubly constrained gravity model

Q3

2016年10月22日 22:52

a) $V_{auto} = -0.01 - 0.05 \times 5 - 0 - 0.03 \times 25 - 0.014 \times 150 = -3.11$

$V_{bus} = -0.07 - 0.05 \times 10 - 0.04 \times 15 - 0.03 \times 40 - 0.014 \times 100 = -3.77$

$$P_{auto} = \frac{e^{V_{auto}}}{e^{V_{auto}} + e^{bus}} = 66\%$$

$$P_{bus} = \frac{e^{bus}}{e^{V_{auto}} + e^{bus}} = 34\%$$

b)

$V_2 = -0.01 - 0.05 \times 5 - 0 - 0.03 \times 25 - 0.014 \times 180 = 3.53$

$$P_{auto} = \frac{e^{V_2}}{e^{V_2} + e^{bus}} = 56\%$$

$$P_{bus} = \frac{e^{bus}}{e^{V_2} + e^{bus}} = 44\%$$

Signal design

STEP 1	Left turn protected or not ?						
	(a) $V_{LT} < 200$ vph OR						
	(b) $V_{LT} \times \frac{\text{Opposing TH}}{\text{No. of lanes}} < 50{,}000$ vph						
	(a) Vlt	Oppo	# of lanes	(b)	(a)-200	(b)-50000	
EB	210	750	2	78750	10	28750	YES
WB	350	775	2	135625	150	85625	YES
SB	15	265	1	3975	-185	-46025	NO
NB	30	250	1	7500	-170	-42500	NO

STEP 2

TCU Volumes

Approach	Movement	Volume (vph)	Equivalent	Volume (tcu)	Shared Lane Grp (tcu)	No. lanes		
EB	LEFT	210	1.05	221	221	1	protected	
	THROUGH	775	1	775	834	2		
	RIGHT	45	1.32	59				
WB	LEFT	350	1.05	368	368	1		
	THROUGH	750	1	750	816	2		
	RIGHT	50	1.32	66				
SB	LEFT	15	3.125	47	325	1	Permited	table 18.1
	THROUGH	265	1	265				
	RIGHT	10	1.32	13				
NB	LEFT	30	3.3125	99	382	1		
	THROUGH	250	1	250				
	RIGHT	25	1.32	33				

Left-turn

Table 18.1: Through Vehicle Equivalents for Left-Turning Vehicles, E_{LT}

Opposing Flow V_o (veh/h)	Number of Opposing Lanes, N_o: 1	2	3
0	1.1	1.1	1.1
200	2.5	2.0	1.8
400	5.0	3.0	2.5
600	10.0*	5.0	4.0
800	13.0*	8.0	6.0
1,000	15.0*	13.0*	10.0*
≥1,200	15.0*	15.0*	15.0*

E_{LT} for all *protected* left turns = 1.05

*indicates that the LT capacity is only available through "sneakers."

Right-turn

Table 18.2: Through Vehicle Equivalent for Right-Turning Vehicles, E_{RT}

Pedestrian Volume In Conflicting Crosswalk (peds/h)	Equivalent
None (0)	1.18
Low (50)	1.21
Moderate (200)	1.32
High (400)	1.52
Extreme (800)	2.14

Signal design

Step 3 Design the signal phasing plan

ϕ_A	221	368	$V_{cA} = 368$ tcu's/ln
ϕ_B	417	408	$V_{cB} = 417$ tcu's/ln
ϕ_C	382	7 325	$V_{cC} = 382$ tcu's/ln

Critical Volume $V_c = 368 + 417 + 382 = 1167$ tcu's

3 phases ; assume loss time is 3 seconds

Step 4. min cycle-length

$$C_{min} = \frac{N \cdot t_L}{1 - \frac{V_c/PHF}{(3600/h)\cdot(v/c)}} = \frac{3 \times 3}{1 - \frac{1167}{1615 \times 0.98 \times 0.95}} = 40.2 \text{ sec} \approx 45 \text{ sec}$$

Step 5 allocation of green time

Total Effective green time $= 45 \text{ sec} - 3 \times 3 \text{ sec} = 36 \text{ sec}$

Then,

$G_A = 368 \times 36 \approx 11$ sec

$G_B = 417 \times 36 \approx 13$ sec

$G_C = 382 \times 36 \approx 12$ sec

Appendix 1

2016年9月18日 17:55

导数公式

一、求导法则

1. 四则运算求导法则

$(f \pm g)' = f' \pm g'$

$(fg)' = f'g + g'f$

$\left(\frac{f}{g}\right)' = \frac{f'g - g'f}{g^2}$

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2. 反函数求导法则

设
$x = f^{-1}(y)$
是
$y = f(x)$
的反函数，则

$[f^{-1}(y)]' = \frac{1}{f'(x)}$

3. 复合函数求导法则

设
$y = f(g(x)), u = g(x),$
则

$\frac{dy}{dx} = \frac{dy}{du} \cdot \frac{du}{dx}$

4. 参数函数求导法则

设
$\begin{cases} x = & \phi(t) \\ y = & \varphi(t) \end{cases},$
则

$\frac{dy}{dx} = \frac{dy/dt}{dx/dt} = \frac{\varphi'(t)}{\phi'(t)}$

5. 对数求导法

如果涉及多项相乘、相除、开方、乘方的情况，可以先取对数再求导.
假设
$y = f(x).$
于是
$\ln y = \ln f(x),$
则
$\frac{y'}{y} = [\ln f(x)]'$

6. 幂指函数求导法

设
$y = u(x)^{v(x)},$
则可采用上述对数求导法有：
$\ln y = v(x) \ln u(x),$
于是

$\frac{y'}{y} = v'(x) \ln u(x) + v(x) \frac{u'(x)}{u(x)}$

或化为指数函数
$u(x)^{v(x)} = e^{v(x) \ln u(x)}$
然后求导.

7. 隐函数求导法则

设
$e^y - \varepsilon \sin xy + \ln \pi = 2016$
确定了
y
关于
x
的函数，则
$y'e^y - \varepsilon(y + xy') \cos(xy) = 0$
于是

$y' = \frac{\varepsilon y}{e^y - \varepsilon x \cos(xy)}$

二、基本初等函数求导公式

$C' = 0$

$(x^\mu)' = \mu x^{\mu - 1}$

$(a^x)' = a^x \ln a, (e^x)' = e^x$

$(\log_a x)' = \frac{1}{x \ln a}, (\ln x)' = \frac{1}{x}$

$(\sin x)' = \cos x$

$(\cos x)' = -\sin x$

$(\tan x)' = \sec^2 x = \frac{1}{\cos^2 x}$

$(\cot x)' = -\csc^2 x = -\frac{1}{\sin^2 x}$

$(\sec x)' = \sec x \cdot \tan x$

$(\csc x)' = -\csc x \cdot \cot x$

$[\ln(x + \sqrt{x^2 + 1})]' = \frac{1}{\sqrt{x^2 + 1}}$

$[\ln(x + \sqrt{x^2 - 1})]' = \frac{1}{\sqrt{x^2 - 1}}$

$(\arcsin x)' = \frac{1}{\sqrt{1 - x^2}}$

$(\arccos x)' = -\frac{1}{\sqrt{1 - x^2}}$

$(\arctan x)' = \frac{1}{1 + x^2}$

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$(\text{arccot } x)' = -\frac{1}{1 + x^2}$

三、高阶导数

$(u \pm v)^{(n)} = u^{(n)} \pm v^{(n)}$

$$(uv)^{(n)} = \sum_{k=0}^{n} C_n^k u^{(n-k)} v^{(k)}$$

$$(\sin kx)^{(n)} = k^n \sin\left(kx + \frac{n\pi}{2}\right)$$

$$(\cos kx)^{(n)} = k^n \cos\left(kx + \frac{n\pi}{2}\right)$$

$$(a^x)^{(n)} = a^x \ln^n a$$

$$\ln^{(n)} x = (-1)^{n-1} \frac{(n-1)!}{x^n}$$

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nchrp 365 report table 6

2016年10月22日 22:52

TABLE 6 Average daily person trips per household by persons per household and auto ownership

Autos Owned	Persons per Household 1	2	3	4	5+	Weighted Average
Urbanized Area Size = 50,000 - 199,999						
Zero	2.6	4.8	7.4	9.2	11.2	3.9
One	4.0	6.7	9.2	11.5	13.7	6.3
Two	4.0	8.1	10.6	13.3	16.7	10.6
Three Plus	4.0	8.4	11.9	15.1	18.0	13.2
Weighted Average	3.7	7.6	10.6	13.6	16.6	9.2
Urbanized Area Size = 200,000 - 499,999						
Zero	2.1	4.0	6.0	7.0	8.0	3.1
One	4.3	6.3	8.8	11.2	13.2	6.2
Two	4.3	7.5	10.6	13.0	15.4	10.1
Three Plus	4.3	7.5	13.0	15.3	18.3	13.5
Weighted Average	3.7	7.1	10.8	13.4	15.9	9.0
Urbanized Area Size = 500,000 - 999,999						
Zero	2.5	4.4	5.6	6.9	8.2	3.4
One	4.6	6.7	8.8	11.0	12.8	6.4
Two	4.6	7.8	10.4	13.0	15.4	10.3
Three Plus	4.6	7.8	12.1	14.6	17.2	12.9
Weighted Average	4.0	7.3	10.2	13.0	15.4	8.7
Urbanized Area Size = 1,000,000+						
Zero	3.1	4.9	6.6	7.8	9.4	4.1
One	4.6	6.7	8.2	10.5	12.5	6.3
Two	4.6	7.8	9.3	11.8	14.7	9.7
Three Plus	4.6	7.8	10.5	13.3	16.2	11.8
Weighted Average	4.2	7.3	9.3	12.0	14.8	8.5

单位

2016年9月19日　　13:42

长度

1英里=5280英尺=1.6093千米

1 Mile=5280 feet=1.6093km

1码=3英尺=0.9144米

1yard=3feet=0.9144m

1英尺=12英寸=0.3048米

1foot=12inches=0.3048m

1英寸=25.4毫米

1inch=25.4mm

面积：

1英尺2=929.03厘米2=0.09290m2

1square feet=929.03cm2=0.09290m2

体积：

1加仑=3.7853公斤

1 gallon=3.7853L

1英尺3=28.32公斤

1 cubic feet=28.32L

1桶=42加仑=158.98公斤

1 barrel=42 gallon=158.98 L

1夸脱液体=1.136公斤

1 quart=1.136L

重量：

1磅=0.4536千克

1pound=0.4536kg

1短吨=2000磅=907.19千克

1 short ton=2000pound=907.19kg

1长吨=2240磅=1016.05千克

1 long ton=2240pound=1016.05kg

压力：

1磅/英寸2=0.070307公斤/平方厘米=6.897千帕

1 psi=0.070307kg/cm2=6.897kpa

流速：

1英尺/秒=0.3048米/秒

1 foot per second=0.3048m/s

流量：

1桶/小时=0.15898米3/时

1 Barrel per hour=0.15898m3/h

1加仑/分0.2271米3/时

1GPM或gpm=0.2271m3h

Table E.2 Area, Length, and Volume Conversion Factors

Quantity	*From Inch-Pound Units*	*To Metric Units*	*Multiply By*
Length	mile	km	1.609344
	yard	m	0.9144
	foot	m	0.3048
		mm	304.8
	inch	mm	25.4
	square mile	km^2	2.59000
Area	acre	m^2	4046.856
		ha (10,000 m^2)	0.4046856
	square yard	m^2	0.83612736
	square foot	m^2	0.09290304
	square inch	mm^2	645.15
	acre foot	m^3	1233.49
Volume	cubic yard	m^3	0.764555
	cubic foot	m^3	0.0283168
	cubic foot	cm^3	28,316.85
	cubic foot	L (1000 cm^3)	28.31685
	100 board feet	m^3	0.235974
	gallon	L (1000 cm^3)	3.78541
	cubic inch	cm^3	16.387064
	cubic inch	mm^3	16,387.064